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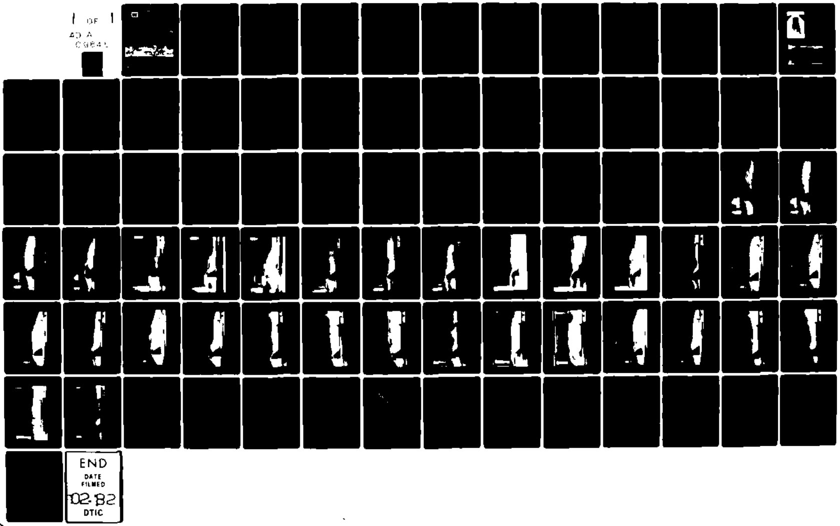

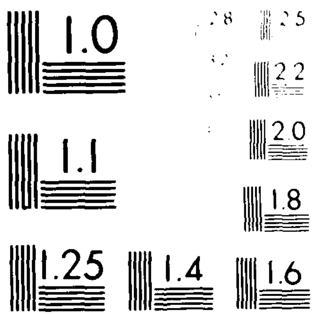
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TECHNICAL REPORT HL-81-13

GRAYS LANDING SPILLWAY AND STILLING BASIN, MONONGAHELA RIVER, PENNSYLVANIA

Hydraulic Model Investigation

by

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Tests were conducted on a 1:36-scale section model to investigate the hydraulic performance of the deeply submerged stilling basin for the Grays Landing Spillway. The model was used to evaluate and develop the most satisfactory stilling basin for optimum hydraulic performance. The purpose of the model investigation also included determining the pressures along the spillway crest, velocities in the stilling basin and exit channel, and the (Continued)

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20. ABSTRACT (Continued)

capability of various stilling basins to pass ice without impacting and abrading the apron, baffle piers, and end sill. The size, gradation, and extent of riprap required for adequate protection of the downstream channel were also determined.

The hydraulic model investigation of the stilling basin revealed the general adequacy of the design of the spillway with satisfactory performance of the proposed spillway crest. From the hydraulic performance of various types of energy dissipators investigated, the most appropriate stilling basin was the type 5 design. Shorter basins produced higher velocities downstream (up to 15.5 fps) and required larger downstream riprap protection. The type 5 design stilling basin allowed the baffle blocks to be located farther downstream to prevent the direct attack of ice against the baffles and stilling basin apron. The recommended type 5 stilling basin design is a conventional hydraulic-jump type energy dissipator which would perform well for both the initial construction of one lock, a 576-ft-wide spillway crest and stilling basin, and the ultimate construction of two locks, a 460-ft-wide spillway crest and stilling basin.

The recommended downstream riprap protection, plan 2, consists of a 72-ft length of d_{100} = 30-in. riprap with a maximum stone weight of 1350 lb, followed by a 108-ft length of d_{100} = 18-in. riprap with a maximum stone weight of 292 lb.

Pressures obtained for a range of flow conditions indicated that no serious negative pressures should be encountered on boundaries of the prototype. Additional analysis of model data was conducted to develop a relationship between headwater, tailwater, and minimum pressures on the upstream quadrant of the spillway crest. For spillway designs subject to submerged flow conditions, the plot presented as Plate 12 herein should be used to select an appropriate design head so that the minimum pressure, considered sufficient to induce cavitation, will not be less than -20 ft.

PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers, U. S. Army, on 5 November 1975, at the request of the U. S. Army Engineer District, Pittsburgh.

The study was conducted during the period December 1975 to February 1978 in the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and under the general supervision of Messrs. J. L. Grace, Jr., Chief of the Hydraulic Structures Division, and N. R. Oswalt, Chief of the Spillways and Channels Branch. The project engineer for the model study was Mr. E. D. Rothwell, assisted by Messrs. B. Perkins and H. Allen. This report was prepared by Messrs. Rothwell, Oswalt, and S. T. Maynord.

During the course of the investigation, Messrs. William Browne and Laszlo Varga of the U. S. Army Engineer Division, Ohio River, and Robert W. Schmitt of the Pittsburgh District visited WES to discuss the program and results of model tests, observe the model in operation, and correlate these results with design studies.

Commanders and Directors of WES during the conduct of the study and the preparation and publication of this report were COL John L. Cannon, CE, COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms

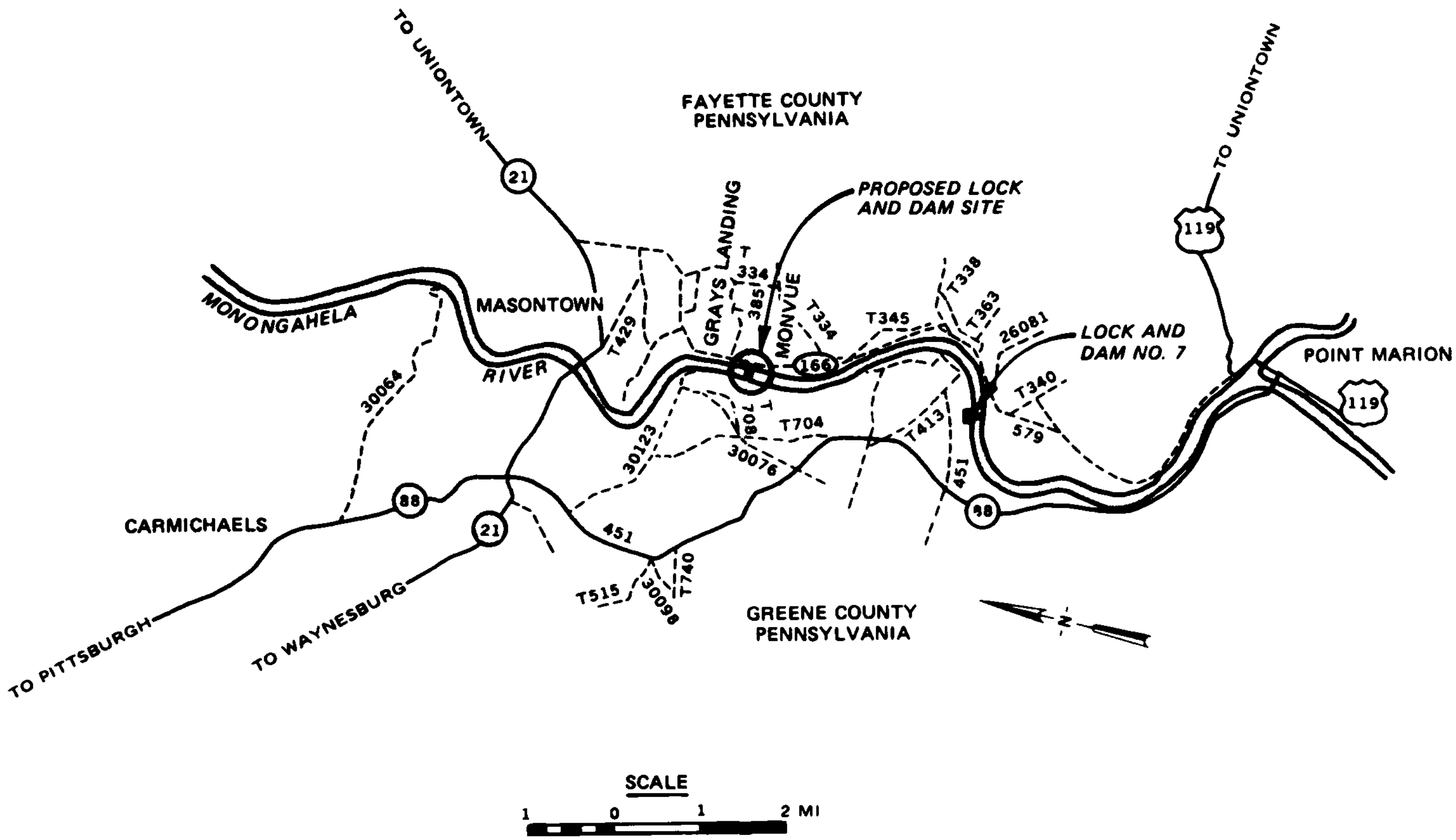

Figure 1. Vicinity map

GRAYS LANDING SPILLWAY AND STILLING BASIN
MONONGAHELA RIVER, PENNSYLVANIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Grays Landing Lock and Dam will be located at river mile 82.2 on the Monongahela River between Fayette and Greene Counties in the vicinity of the community of Grays Landing, Pennsylvania, 2.8 river miles downstream from existing Lock and Dam 7 (Figure 1). The structure will contain an overflow spillway with a maximum height of 28 ft* above the riverbed, a crest elevation of 778.0,** and a length of 576 ft located on the left of the navigation lock (Plate 1).

Purpose of Model Study

2. A section model of the spillway was constructed to investigate hydraulic performance of the deeply submerged stilling basin for the range of expected flow conditions. Specifically, the model study would provide the means necessary to evaluate and develop a stilling basin design that will provide satisfactory hydraulic capacity and energy dissipation. The following information was obtained:

- a. Flow characteristics and stilling basin performance with both a conventional horizontal stilling basin containing two rows of baffle piers and an end sill, and a roller bucket type of energy dissipator.
- b. Pressures along the spillway crest and velocities in the stilling basin and exit channel.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD).

c. Relative capability of various stilling basin designs to pass simulated ice over the weir and through conventional stilling basins without impacting and abrading the apron, baffle piers, and end sill.

d. Size, gradation, and extent of riprap required for adequate protection of the downstream channel.

PART II: THE MODEL

Description

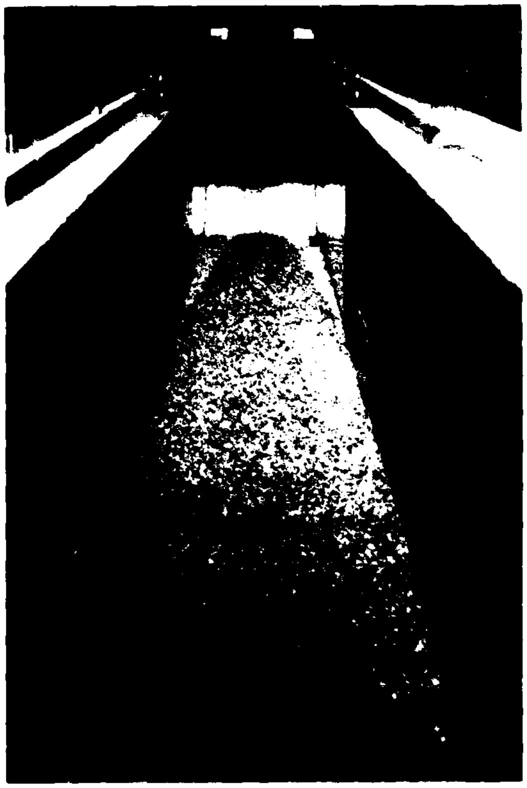
3. A 1:36-scale section model was constructed to simulate a 72-ft-wide portion of the uncontrolled spillway structure, about 1280 ft of approach, and about 1080 ft of exit channel in a 2.0-ft-wide glass-sided flume (Figure 2). Portions of the model representing the approach and exit channels were molded with stone. The ungated ogee weir was fabricated of sheet metal. The conventional horizontal stilling basin and the roller bucket energy dissipator were fabricated with plastic-coated plywood and wood treated with a waterproofing compound to prevent expansion.

4. Water used in the operation of the model was supplied by pumps, and discharges were measured by means of venturi and orifice meters. Steel rails set to grade provided reference planes for measuring devices. Water-surface elevations were obtained by point gages. Velocities were measured with pitot tubes and velocity meters. Pressures were measured by piezometers installed along the center line of the structure.

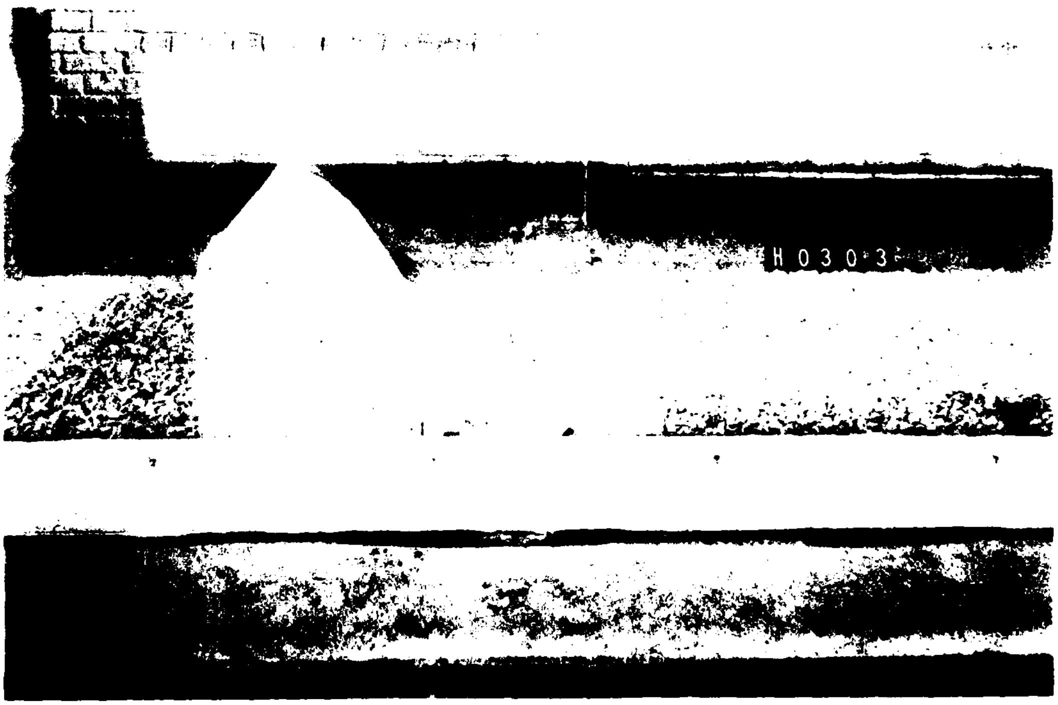
Scale Relations

5. The accepted equations of hydraulic similitude, based upon Froudian criteria, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and prototype. The general relations expressed in terms of the model scale or length ratio, L_r, are presented in the following tabulation:

Dimension	Ratio	Scale Relations
Length	L_r	1:36
Area	$A_r = L_r^2$	1:1,296
Velocity	$V_r = L_r^{1/2}$	1:6
Discharge	$Q_r = L_r^{5/2}$	1:7,776
Time	$T_r = L_r^{1/2}$	1:6



a. Looking upstream; exit channel, structure, and approach channel



b. Spillway and stilling basin

Figure 2. The 1:36-scale section model

6. Model measurements of each dimension or variable can be transferred quantitatively to prototype equivalents by means of the preceding scale relations.

PART III: TESTS AND RESULTS

Spillway Crest

Original design

7. Details of the original design spillway crest are presented in Figure 3. Shapes of the upstream and downstream quadrants were based on a design head of 12 ft and approach depth of 28 ft. The sponsor's design of the upstream quadrant of the crest, as shown in Plate 1 (EM 1110-2-1603),* was replaced during model design with an ellipse described by the equation

$$\frac{X^2}{A^2} + \frac{Y^2}{B^2} = 1$$

where A and B are functions of the ratio P/H_d, where P is the approach depth and H_d is the design head. Details of this design procedure are found in MP H-73-5.** The downstream quadrant was shaped to the equation

$$X^{1.85} = 2H_d^{0.85}Y$$

based on Engineer Manual 1110-2-1603. This is confirmed by the miscellaneous paper. Model tests indicated satisfactory performance of the elliptical spillway crest shape used during the study (Figure 3).

Stilling Basin Performance

8. Tests to evaluate the hydraulic performance of various types

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* Office, Chief of Engineers, Department of the Army. 1965 (Mar). "Hydraulic Design of Spillways," EM 1110-2-1603, Washington, D. C.

** T. E. Murphy. 1973 (Dec). "Spillway Crest Design," Miscellaneous Paper H-73-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

of energy dissipators and to determine the most appropriate stilling basin design were conducted by introducing a full range of discharges associated with various depths of tailwater including those required for significant submergence of the spillway. Velocities observed over the end sill and in the exit channel are presented in Table 1 for the various types of energy dissipators investigated. All stilling basin tests were conducted with a 28-ft approach depth.

Type 1 (original)
design energy dissipator

9. The project details shown in Plate 1 as provided by the sponsor were refined (replaced upstream quadrant, added 10-ft radius toe curve, and a 1-on-1 end sill) to reflect the most desirable features of the more recent findings from spillway research. Model observations with the type 1 (original) design energy dissipator, which consisted of a horizontal apron and a 4-ft-high end sill (Figure 3), indicated that two types of stilling basin action might occur within the range of

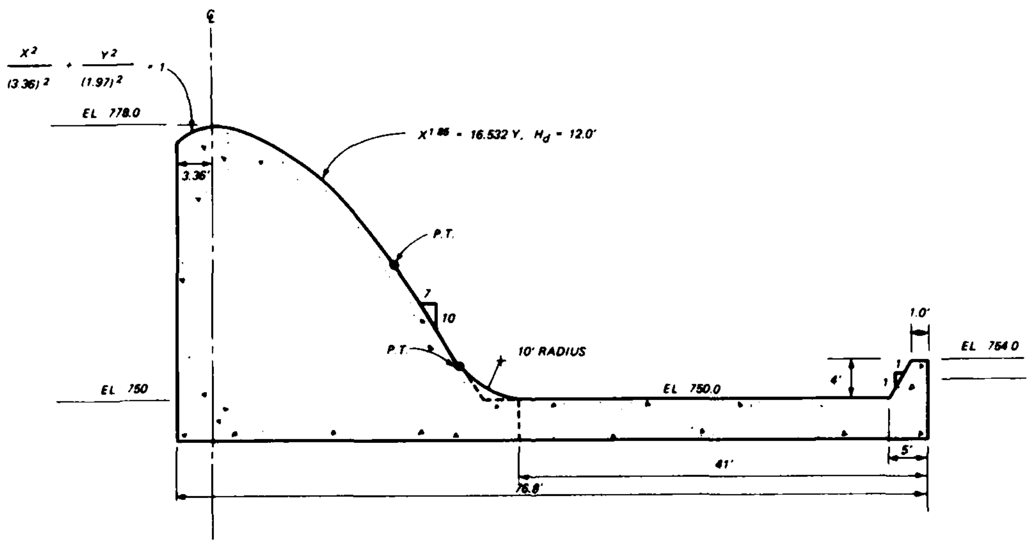

Figure 3. Type 1 (original) design crest and energy dissipator

anticipated discharge and tailwater elevations to which the structure may be subjected. These basin actions are defined and illustrated in Photos 1-4. Headwaters in the range of el 784.9 to 794.7 caused

turbulent wave action in the exit channel and a concentration of flows along the bottom of the apron. Velocities measured at 0.6-ft depth over the end sill and in the downstream exit channel are presented in Table 1.

Types 2 and 3 design
energy dissipators

10. The types 2 and 3 design energy dissipators were based on the conjugate depths to be expected with unit discharges of 106 cfs/ft and 261 cfs/ft, respectively, and represented conventional hydraulic-jump type stilling basins with horizontal aprons and appropriate baffle piers and end sills.

11. Hydraulic performance of the type 2 design, which consisted of a 40.25-ft-long horizontal apron superimposed with two rows of 2.5-ft-high baffle piers and terminated with a 1.25-ft-high end sill (Figure 4),

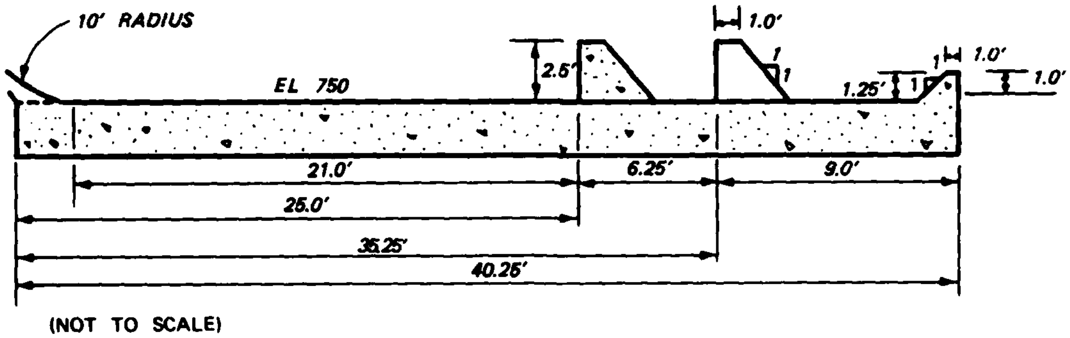

Figure 4. Type 2 design energy dissipator

and the type 3 design, which consisted of a 61-ft-long horizontal apron superimposed with two rows of 5-ft-high baffle piers and terminated with a 2.5-ft-high end sill (Figure 5) was generally adequate for all anticipated flows. However, the magnitudes of velocities measured at the end sill and in the exit channel were sufficiently large to indicate the potential for scour with minimal riprap protection. Velocities for various flow conditions are presented in Table 1. Basin actions for various flow conditions are presented in Photos 5-10.

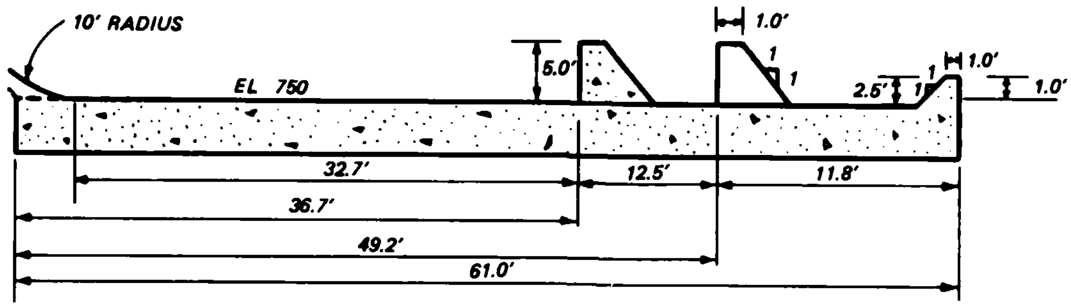

Figure 5. Type 3 design energy dissipator

Type 4 design
energy dissipator

12. The type 4 roller bucket design (Figure 6) consisted of a 10-ft-radius bucket with a 3-ft-high lip that terminated at a 45-deg angle with the horizontal plane. Flow conditions observed at unit

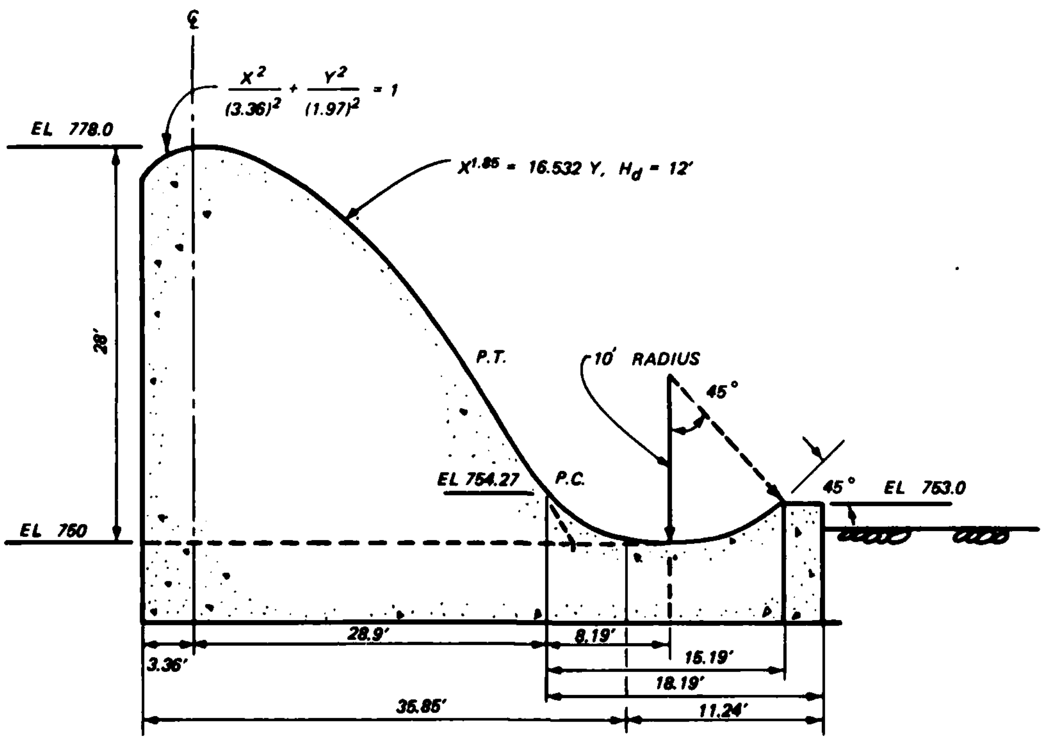

Figure 6. Type 4 design energy dissipator

discharges of 66.7 and 147.5 cfs/ft (Photos 11 and 12) illustrate the turbulent wave action in the exit channel caused by the extremely short roller bucket energy dissipator. The magnitudes of velocities in the vicinity of the lip of the bucket and in the exit channel were significantly greater than those measured with the previous energy dissipator designs for unit discharges ranging from 66.7 to 147.5 cfs/ft and 211.5 to 251.3 cfs/ft (Table 1). These results indicate that the type 4 design roller bucket energy dissipator is less desirable than either the short or long conventional horizontal hydraulic-jump type stilling basins. Basin actions observed with various discharges are shown in Photos 11-13.

13. Due to the unsatisfactory energy dissipation and surface waves observed with the roller bucket design, tests were redirected to develop a conventional hydraulic-jump type stilling basin that would provide adequate energy dissipation and require only minimal riprap protection in the exit channel.

Type 5 (Recommended), Type 6, and Type 7 Design Energy Dissipators

14. Additional tests of several other designs of conventional horizontal hydraulic-jump type stilling basins resulted in a hydraulically favorable stilling basin design (type 5) whose details are shown in Plate 2. Performance of the type 5 design stilling basin was adequate for all anticipated flow conditions. Basin actions for various flow conditions are presented in Photos 14-20.

15. The types 5 and 6 stilling basin designs are identical except for the height of the baffle piers which are 9 ft and 5 ft, respectively. The type 7 stilling basin design is also identical to the type 5 stilling basin design except that baffle piers in the first row are 5 ft high and in the second row are 9 ft high, as suggested by the Pittsburgh District. Velocities measured at two locations downstream of the types 5 to 7 stilling basin designs indicate that all three basin designs produce similar energy dissipation with maximum velocities of 9.6 fps for types 5 and 7 and 10.7 fps for type 6. Velocities for various discharges

are presented in Table 1. Basin actions for various flow conditions with the types 6 and 7 stilling basin designs are presented in Photos 21-26.

Ice Passage

16. Tests were conducted with the types 5 to 7 stilling basin designs to determine if the two rows of baffle piers would be vulnerable to abrasive erosion from ice passage during winter operations. A low-density polyethylene material that has the same density as ice was used to simulate 14.4- by 14.4-ft blocks of ice nominally 2 and 4 ft thick. Results of the model tests demonstrate that in the recommended design positions, the baffle piers and the stilling basin apron would not be subjected to damage by ice passage. Basin performance with various flow conditions is presented in Photos 27-32.

Riprap Protection

17. Riprap stability tests were conducted downstream of the types 5 and 6 design stilling basins to determine the most adequate riprap protection plan below the structure for the anticipated range of flow conditions. The type 7 basin produced similar energy dissipation and downstream velocity as did type 5 and therefore the riprap protection for type 5 was considered adequate. Results of the riprap stability tests with plans 1 and 2 for the types 5 and 6 design stilling basins are presented along with a description of the riprap plans in Tables 2 and 3. Based on visual observations of the riprap stability tests, the type 5 stilling basin design with 9-ft-high baffle piers (Plate 2) and riprap protection plan 2 is recommended. The recommended plan of riprap consists of a 72-ft length of stone with a d_{100} = 30-in. riprap and a maximum weight of 1350 lb, followed by a 108-ft length of d_{100} = 18-in. riprap with a maximum stone weight of 292 lb which should be adequate for all expected operating conditions.

Discharge Characteristics

Flow conditions

18. Tests to determine the discharge characteristics of the ungated spillway structure with expected maximum and minimum approach depths of 28 and 12 ft were conducted for both free and submerged flow conditions. The upstream quadrant crest shape was based on an approach depth $P = 28$ ft and $h_d = 12$ ft for both approach depths.

Description of tests

19. Tests to determine the discharge characteristics of the structure for free uncontrolled flows were conducted by introducing various discharges into the model, with the tailwater below the spillway crest, and observing the corresponding upper pool elevations. Sufficient time was allowed for stabilization of the upstream flow conditions.

20. Submerged flow discharge characteristics for uncontrolled flows considered independent of stilling basin type were determined by introducing several constant discharges into the model and varying the tailwater by small increments for each from an elevation at which no interference in spillway flow was evident to an elevation at which the flow was practically 100 percent submerged. The elevation of the upper pool was noted at each of the respective tailwater elevations.

Weir capacity

21. The head-unit discharge rating curves for free uncontrolled flow over the spillway with approach depths of 28 ft and 12 ft are presented in Plate 3. The equations for these curves are the best empirical fit of the data by the method of least squares. The following equations satisfy the basic calibration data obtained in the model:

$$q = 2.795 H_e^{1.64} \quad \text{(28-ft approach depth)}$$

and

$$q = 2.928 H_e^{1.60} \quad \text{(12-ft approach depth)}$$

where

q = unit discharge, cfs

H_e = total head on crest (including approach velocity head), ft

22. Comparisons of the model and computed spillway rating curves for a 28-ft approach depth are presented in Plate 4 for weir lengths of 576 ft and 460 ft. The two weir lengths represent initial and future crest lengths with provision for single and double locks, respectively. Both model and computed rating curves were obtained by including abutment contractive effects in the general weir formula $Q = CLH_e^{3/2}$, where C is the discharge coefficient as indicated by Corps of Engineers Hydraulic Design Chart (HDC) 111-3;* H_e is the energy head above the weir crest in feet; L is the effective length of spillway determined from the expression $L = L' - 2KH_e$, in which L' is the net length of crest in feet; and K is the abutment contraction coefficient determined from HDC 111-3/1. In addition, computed curves allowed for expansive flows over lock walls at higher stages. The model indicates less efficiency than the prototype because the adjustment was not made for expansive flows.

Calibration data

23. The basic calibration data, presented in Plates 5-7, show the approach channel energy elevation (water surface plus velocity head based on average velocity) corresponding to a particular elevation of the tailwater for a given discharge observed with approach depths of 28 and 12 ft.

24. Free and submerged uncontrolled flow data for approach depths of 28 and 12 ft are shown in Plates 5 and 6, and 7, respectively. The data for each of the various discharges shown in the respective plates illustrate the following:

- a. The relation between the elevation of the energy of flow in the approach channel and the elevation of the tailwater in the exit channel.

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* U. S. Army, Corps of Engineers, "Hydraulic Design Criteria," prepared for Office, Chief of Engineers, by U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss., issued serially since 1952.

b. The range of tailwater elevations that do not affect flow and for which the elevation of the approach flow energy is constant, i.e. the range of free uncontrolled flow.

c. The range of tailwater elevations that do affect flow and for which the elevation of the approach flow energy is controlled by the submergence effect of the tailwater, i.e. the range of submerged uncontrolled flow.

Analyses of data

25. The empirical equations that satisfy the experimental data for free and submerged uncontrolled flows are as follows:

a. Free uncontrolled flow:

$Q = CLH_e^{3/2}$, where C is a function of H_e/H_d

b. Submerged uncontrolled flow:

$Q = C_s Lh\sqrt{2g\Delta H}$, where C_s is a function of h/H_e

Symbols used in these equations are defined as follows:

Q = discharge, cfs

C = discharge coefficient for free uncontrolled flow

C_s = discharge coefficient for submerged uncontrolled flow

L = length of weir crest, ft

h = tailwater elevation referred to weir crest, ft

H_e = total energy head on weir crest, ft

H_d = design head on crest, ft

g = acceleration due to gravity, ft/sec^2

ΔH = difference between total energy head of flow in the approach channel and elevation of tailwater with reference to the spillway crest $(H_e - h)$, ft

Uncontrolled flow discharge coefficients

26. Quantities determined from the experimental data were substituted in the above equations, and the discharge coefficients for the respective flow regimes were computed. Free uncontrolled flow discharge coefficients for the equation $Q = CLH_e^{3/2}$ and approach depths of 28 and 12 ft are presented in Plate 8. Submerged uncontrolled flow discharge

coefficients for the equation $Q = C_s Lh \sqrt{2g\Delta H}$, various degrees of submergence, and approach depths of 28 ft and 12 ft are presented in Plate 9.

Flow regimes

27. Model data were analyzed to define the limits of each flow regime and corresponding discharge equation. An investigation of the basic data obtained with a constant discharge and uncontrolled flow reveals that there is a tailwater elevation at which the energy of the approach channel flow increases with a corresponding increase in the tailwater. This is the elevation at which the tailwater begins to submerge the flow, and free flow becomes submerged flow. Results of analyses to distinguish between free and submerged uncontrolled flows with approach depths of 28 and 12 ft are presented in Plate 10.

Effect of Approach Depth on Discharge Coefficients

28. Effects of the depth of approach on the free and submerged flow discharge coefficients with the exit channel elevation fixed 28 ft below the spillway crest are shown in Plates 8 and 9. In general, these data indicate that for a fixed exit channel elevation, the greater the depth of approach, the greater the value of the free and/or submerged flow discharge coefficients. Part of the reason for this is that the crest shape for both approach depths was based on a 28 ft approach depth.

Pressures on Structure

29. Piezometers, located as shown in Plate 11, were used to determine the hydrostatic pressures along the center line of the spillway crest. Pressures obtained with various flow conditions are shown in Tables 4-7. Although only a limited number of pressures were measured during the investigation, these data are representative of the range of flow conditions expected and indicate that no serious negative pressures should be encountered on the proposed spillway.

30. Additional tests and data analyses were conducted to develop generalized relations between headwater, tailwater, and minimum pressures on the upstream quadrant of the spillway crest. The ratio of minimum pressures (hp) on upstream quadrant to design head (H_d) for various ratios of H_e/H_d and submergence h/H_e, where H_d is design head and h is depth of tailwater above the weir crest, are presented in Plate 12. Minimum pressures obtained with various flow conditions are presented in Table 8. In the design of spillways, it is recommended that a ratio of H_e/H_d be selected such that the minimum pressure will never be less than -20 ft.* Results presented in Plate 12 can be used to select an appropriate design head for spillways subject to submerged flow conditions.

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* E. S. Melsheimer and T. E. Murphy. 1970 (Jan). "Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway," Research Report H-70-1, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

PART IV: SUMMARY

31. The hydraulic model investigation of the Grays Landing spillway and stilling basin revealed the general adequacy of the design of the spillway. Model tests indicated satisfactory performance of the elliptical spillway crest (Figure 3) used during the study.

32. The hydraulic performance of various types of energy dissipators was investigated to determine the most appropriate stilling basin design. Based on model results, the type 5 design stilling basin is recommended. Shorter basins produced higher velocities downstream (up to 15.5 fps) which would require larger downstream riprap protection. The type 5 design stilling basin allows the baffle blocks to be located farther downstream to prevent the direct attack of ice against the baffles. A detailed analysis of the resistance of the natural rock downstream of the subject spillway to various Froude numbers of flow could indicate adequate stability without the need for riprap with the lower velocities of 9.5 fps and the type 5 design stilling basin. However, such information was not available during the study. The type 4 design energy dissipator (a roller bucket) was less desirable than either a relatively short or long conventional hydraulic-jump type stilling basin due to higher velocities (up to 19 fps) and exiting Froude numbers of flow.

33. Stilling basin type 5 and riprap plan 2 are recommended based on visual observations of the riprap tests and results of the hydraulic tests. The recommended type 5 stilling basin design is a conventional hydraulic-jump type energy dissipator with two rows of baffles and an end sill which would perform well for both the initial construction of one lock with a 576-ft-wide crest and stilling basin and the ultimate construction of two locks and a 460-ft-wide crest and stilling basin. The recommended riprap protection, plan 2, consists of a 72-ft length of d_{100} = 30-in. riprap with a maximum stone weight of 1350 lb, followed by a 108-ft length of d_{100} = 18-in. riprap with a maximum stone weight of 292 lb.

34. Results of tests to determine the discharge characteristics

of the ungated spillway indicated that discharge characteristics of the two possible flow regimes can be satisfied by the following equations:

a. Free uncontrolled flow:

$$Q = CLH_e^{3/2} \text{ , where } C \text{ is a function of } H_e/H_d$$

b. Submerged uncontrolled flow:

$$Q = C_s Lh\sqrt{2g\Delta H} \text{ , where } C_s \text{ is a function of } h/H_e$$

Discharge coefficients applicable to each of these flow conditions and equations are shown in the respective plates relating the coefficients and the pertinent variables and parameters. The limits of each flow regime and the corresponding discharge equation are shown in graphic plots in terms of dimensionless quantities.

35. In general, the effects of approach depth on free and submerged flow discharge coefficients indicate that with a fixed exit channel elevation, the greater the depth of approach, the greater the value of the free and submerged flow discharge coefficients.

36. Pressures obtained for a range of flow conditions indicated that no serious negative pressures should be encountered on boundaries of the prototype. Additional analysis of model data was conducted to develop a relationship between headwater, tailwater, and minimum pressures on the upstream quadrant of the spillway crest. For designs subject to submerged flow conditions, the plot presented as Plate 12 herein should be used to select an appropriate design head so that the minimum pressure will not be less than -20 ft. Pressures less than -20 ft will likely produce cavitation.

Table 1

Velocities with Various Designs of Energy Dissipators

Test No.	HW	TW	q cfs/ft	Type 1 End Sill	Type 1 130 ft* Downstream	Type 2 End Sill	Type 2 130 ft* Downstream	Type 3 End Sill	Type 3 130 ft* Downstream	Type 4 End of Lip	Type 4 130 ft* Downstream
1	780.9	764.5	16.3	4.8	3.4	3.4	3.0	4.8	3.0	8.3	2.7
2	782.4	766.7	32.6	5.9	4.3	4.8	4.0	3.4	3.7	5.9	3.9
3	783.9	770.0	51.6	6.8	5.1	4.8	4.4	3.4	4.9	5.9	5.4
4	784.9	772.3	66.7	6.8	5.9	8.1	4.8	4.8	5.2	15.9	6.1
5	786.3	774.7	89.6	6.8	4.8	9.6	4.8	5.9	5.7	11.8	6.3
6	787.2	776.8	105.6	5.9	6.4	9.6	5.6	5.9	6.1	14.5	6.6
7	787.9	778.7	119.6	6.8	7.2	9.6	6.5	5.9	6.6	10.7	7.4
8	789.2	780.8	147.5	6.8	6.8	9.0	7.3	6.8	7.4	4.8	8.0
9	789.6	782.4	153.3	6.8	6.8	6.8	7.6	7.6	7.2	3.4	7.8
10	790.7	784.5	179.2	4.8	6.5	8.3	7.0	10.2	8.3	3.4	7.9
11	792.5	787.4	211.5	8.4	8.4	5.9	8.2	13.2	9.6	16.0	10.2
12	794.7	789.7	251.3	9.6	10.2	14.4	9.6	14.5	11.6	19.8	10.8
13	797.5	793.7	280.5	15.9	12.7	16.0	10.8	15.5	12.6	17.3	12.6
14	799.5	796.4	296.0	15.2	11.3	16.0	11.5	14.8	12.4	12.7	12.0
15	802.2	799.2	328.8	15.2	11.8	11.8	10.1	14.1	9.3	11.8	9.0

(Continued)

Note: 28-ft approach depth, velocities at 0.6 × depth below water surface.
Headwater (HW) and tailwater (TW) do not account for expansive flow over lock wall.
HW includes velocity head.

* Velocity measured 130 ft downstream from weir crest.

Table 1 (Concluded)

Test No.	HW	TW	q cfs/ft	Type 5 End Sill	Type 5 180 ft* Downstream	Type 6 End Sill	Type 6 180 ft* Downstream	Type 7 End Sill	Type 7 180 ft* Downstream
1	780.9	764.5	16.3	0.0	3.4	0.0	3.4	0.0	3.4
2	782.4	766.7	32.6	0.0	3.4	0.0	3.4	0.0	3.4
3	783.9	770.0	51.6	0.0	3.4	0.0	3.4	0.0	3.4
4	784.9	772.3	66.7	2.6	3.4	0.0	3.4	0.0	3.4
5	786.3	774.7	89.6	2.6	3.4	0.0	3.4	0.0	3.4
6	787.2	776.8	105.6	2.6	3.4	0.0	3.4	2.1	3.4
7	787.9	778.7	119.6	2.6	3.4	2.1	3.4	2.1	3.4
8	789.2	780.8	147.5	2.6	3.4	2.1	3.4	2.1	3.4
9	789.6	782.4	153.3	2.1	3.4	2.1	3.4	2.1	3.4
10	790.7	784.5	179.2	2.1	3.4	2.1	3.4	2.1	3.4
11	792.5	787.4	211.5	2.1	3.4	4.3	4.8	2.1	4.8
12	794.7	789.7	251.3	7.6	6.8	10.7	8.3	9.6	9.0
13	797.5	793.7	280.5	9.6	9.0	10.7	9.6	9.6	9.6
14	799.5	796.4	296.0	8.3	8.3	8.3	8.3	7.6	9.0
15	802.2	799.2	328.8	6.8	9.6	6.8	8.3	5.3	9.0

Note: 28-ft approach depth, velocities at 0.6 × depth below water surface.
Headwater (HW) and tailwater (TW) do not account for expansive flow over lock wall.
HW included velocity head.
* Velocity measured 180 ft downstream from weir crest.

Table 2

Stability of Downstream Riprap Protection with Type 5 Stilling Basin Design

HW	TW	Stability of Riprap Protection Plan 1	Plan 2
783.5	766.7	Stable	Stable
	765.8	Stable	Stable
	764.4	Stable	Stable
	763.2	Stable	Stable
786.5	772.3	Stable	Stable
	771.6	Stable	Stable
	769.8	Stable	Stable
	768.0	Stable	Stable
	766.2	Stable	Stable
788.6	776.8	Stable	Stable
	775.2	Stable	Stable
	773.4	Stable	Stable
	771.6	Stable	Stable
	769.8	Stable	Stable
	768.0	Failure	Stable
790.7	780.8	Stable	Stable
	778.8	Stable	Stable
	777.0	Stable	Stable
	775.2	Stable	Stable
	773.4	Stable	Stable
	771.6	Stable	Stable
	769.6	Failure	Stable
792.8	784.5	Stable	Stable
	782.4	Stable	Stable
	780.6	Stable	Stable
	778.8	Stable	Stable
	777.0	Stable	Stable
	775.2	Stable	Stable
	773.4	Failure	Stable
794.6	787.4	Stable	Stable
	785.5	Stable	Stable
	783.5	Stable	Stable
	781.7	Stable	Stable
	779.9	Stable	Stable
	778.0	Stable	Stable
	776.3	Failure	Stable

(Continued)

Note: Headwater includes velocity head.
Plan 1 consists of 72 ft of riprap, with a maximum stone weight of 691 lb (d_{100} = 24 in.), and followed by 108 ft of riprap, with a maximum stone weight of 292 lb (d_{100} = 18 in.).
Plan 2 consists of 72 ft of riprap, with a maximum stone weight of 1350 lb (d_{100} = 30 in.), and followed by 108 ft of riprap, with a maximum stone weight of 292 lb (d_{100} = 18 in.).

Table 2 (Concluded)

HW	TW	Stability of Riprap Protection Plan 1	Plan 2
796.8	789.7	Stable	Stable
796.8	787.8	Stable	Stable
796.8	786.0	Stable	Stable
796.9	784.2	Stable	Stable
796.9	782.0	Failure	Stable
798.5	793.7	Stable	Stable
798.6	791.4	Stable	Stable
798.6	790.3	Failure	Stable
798.7	789.2	Failure	Stable
800.3	796.4	Failure	Stable
800.4	795.3	Failure	Stable
800.5	794.0	Failure	Stable
800.6	791.2	Failure	Stable
803.1	797.0	Failure	Stable
803.1	790.0	Failure	Failure
803.1	788.0	Failure	Failure

Table 3

Stability of Downstream Riprap

Protection with Type 6 Stilling Basin Design

HW	TW	Stability of Riprap Protection Plan 1	Plan 2
783.5	766.7	Stable	Stable
	765.0	Stable	Stable
	764.1	Stable	Stable
	763.0	Stable	Stable
	762.0	Stable	Stable
786.5	772.3	Stable	Stable
	766.3	Stable	Stable
	764.9	Stable	Stable
	762.0	Stable	Stable
788.6	776.8	Stable	Stable
	767.8	Stable	Stable
	765.2	Stable	Stable
	764.2	Stable	Stable
790.7	780.8	Stable	Stable
	771.6	Stable	Stable
	769.5	Stable	Stable
	767.5	Stable	Stable
792.8	784.5	Stable	Stable
	775.0	Stable	Stable
	773.0	Stable	Stable
	771.0	Stable	Stable
	768.0	Failure	Stable
794.6	787.4	Stable	Stable
	775.9	Stable	Stable
	774.0	Stable	Stable
	772.0	Failure	Stable
796.8	789.7	Stable	Stable
796.9	782.2	Failure	Stable
796.9	780.0	Failure	Failure
796.9	778.0	Failure	Failure

(Continued)

Note: Plan 1 consists of 72 ft of riprap, with a maximum stone weight of 691 lb (d_{100} = 24 in.), and followed by 108 ft of riprap, with a maximum stone weight of 292 lb (d_{100} = 18 in.).
Plan 2 consists of 72 ft of riprap, with a maximum stone weight of 1350 lb (d_{100} = 30 in.), and followed by 108 ft of riprap, with a maximum stone weight of 292 lb (d_{100} = 18 in.).

Table 3 (Concluded)

HW	TW	Stability of Riprap Protection Plan 1	Plan 2
798.6	791.3	Stable	Stable
798.9	790.0	Failure	Stable
799.0	788.0	Failure	Stable
799.0	786.0	Failure	Stable
799.0	784.0	Failure	Failure
800.5	794.0	Failure	Stable
800.6	792.0	Failure	Failure
800.8	790.0	Failure	Failure
803.1	797.0	Failure	Failure
803.1	795.5	Failure	Failure

Table 4

Pressures on Spillway Crest

Pressure in Prototype Feet of Water

Crest Length 576 ft; Approach Depth 28 ft

Piezometer No.	Piezometer El	HW = 780.9 TW = 764.5	HW = 782.4 TW = 766.6	HW = 783.9 TW = 770.0	HW = 784.9 TW = 772.3	HW = 786.3 TW = 774.7	HW = 787.2 TW = 776.8	HW = 787.9 TW = 778.7	HW = 789.2 TW = 780.8
1	768	13.5	15.0	16.4	17.5	19.0	19.8	20.7	22.0
2	773	8.5	10.0	11.4	12.3	13.8	14.5	15.3	16.7
3	776	5.3	6.0	6.5	6.8	6.5	6.2	6.0	5.3
4	777.8	2.7	3.2	3.2	3.2	3.0	2.7	2.2	1.2
5	778	2.3	3.0	3.0	3.2	3.0	3.0	2.5	2.0
6	777.6	2.2	2.6	3.2	3.2	3.2	3.1	2.6	2.4
7	777	1.8	2.2	2.5	2.8	2.5	2.5	2.5	2.0
8	774	2.0	1.5	1.8	1.8	2.0	1.8	2.0	2.5
9	771	1.0	1.0	1.0	1.2	1.5	2.0	2.8	4.0
10	767.5	1.3	1.3	1.3	2.0	3.5	4.7	6.3	8.5
11	763	1.2	2.0	4.0	6.0	8.0	9.5	11.2	14.0
12	759	4.5	6.0	8.8	11.0	13.5	15.8	18.0	21.2
13	755	8.7	10.5	14.5	17.5	21.0	23.5	25.5	28.0
14	753	12.0	15.3	19.5	22.2	25.8	27.5	29.0	30.5
15	752	12.8	15.8	20.5	23.7	27.0	29.0	30.5	32.0
16	750.5	14.5	18.0	22.7	26.0	29.5	31.0	32.5	37.5

(Continued)

Note: Elevations are in feet referred to NGVD. HW = headwater (includes velocity head), TW = tailwater.

Table 4 (Concluded)

Piezometer No.	El	HW = 789.6 TW = 782.4	HW = 790.7 TW = 784.5	HW = 792.5 TW = 787.4	HW = 794.7 TW = 789.7	HW = 797.5 TW = 793.7	HW = 799.6 TW = 796.4	HW = 802.2 TW = 799.2	HW = 812.8 TW = 810.6
1	768	22.2	23.3	24.7	26.3	28.8	31.0	33.0	44.5
2	773	16.5	17.5	19.0	20.5	23.0	25.0	27.5	39.0
3	776	5.0	4.5	4.0	3.0	6.0	10.0	14.0	31.0
4	777.8	1.0	0.2	-0.3	-0.8	3.2	8.2	12.2	29.5
5	778	1.8	1.5	1.2	1.0	5.5	10.5	14.0	29.8
6	777.6	2.4	2.4	3.2	3.6	9.4	13.4	16.4	30.4
7	777	2.5	2.5	3.8	4.5	11.0	16.0	18.0	31.0
8	774	3.5	4.5	7.5	10.0	17.0	20.0	22.5	34.0
9	771	6.0	8.0	11.5	15.0	20.0	23.0	25.5	37.0
10	767.5	10.5	13.3	17.5	20.5	23.5	26.5	29.0	40.5
11	763	16.7	19.5	23.8	25.0	28.0	31.0	33.5	45.0
12	759	23.3	26.0	28.0	29.0	32.0	35.0	37.5	49.0
13	755	29.0	30.5	32.0	32.0	36.0	39.0	41.5	53.0
14	753	31.5	33.0	34.0	35.0	38.0	41.0	43.5	55.0
15	752	33.0	34.5	35.5	36.0	39.0	42.0	44.5	56.0
16	750.5	35.5	36.5	38.0	38.5	40.5	43.5	46.0	57.5

Table 5

Pressures on Spillway Crest

Pressure in Prototype Feet of Water

Crest Length 576 ft; Approach Depth 12 ft

Piezometer No.	Piezometer El	HW = 782.4 TW = 766.7	HW = 785.3 TW = 772.3	HW = 787.5 TW = 776.8	HW = 789.4 TW = 780.8	HW = 791.2 TW = 784.5
1	768	15.8	18.3	20.3	22.0	23.8
2	773	10.8	13.3	15.0	16.8	18.0
3	776	6.5	7.0	7.0	6.3	5.5
4	777.8	3.7	4.0	3.2	2.2	1.2
5	778	3.0	3.5	3.0	2.0	1.5
6	777.6	2.9	3.4	3.4	2.9	3.4
7	777	2.5	3.0	3.0	2.8	4.0
8	774	1.5	2.0	2.0	3.8	6.8
9	771	1.3	1.5	3.5	6.8	10.8
10	767.5	1.5	3.5	7.3	11.0	16.0
11	763	3.5	8.3	12.3	17.0	22.5
12	759	8.5	13.5	18.3	23.8	27.0
13	755	12.8	19.5	25.5	28.8	31.0
14	753	17.0	24.0	28.5	31.0	33.0
15	752	17.0	25.0	29.8	32.5	34.0
16	750.5	20.5	27.5	31.8	34.5	36.0
		HW = 793.0 TW = 787.4	HW = 795.4 TW = 789.7	HW = 798.3 TW = 793.7	HW = 800.2 TW = 796.5	HW = 802.6 TW = 799.2
1	768	25.3	27.0	30.0	31.8	34.5
2	773	19.8	21.5	24.5	26.5	29.0
3	776	5.0	5.0	8.0	11.0	14.0
4	777.8	1.2	0.7	4.7	9.2	13.2
5	778	1.8	2.0	6.0	10.0	15.0
6	777.6	4.4	5.4	10.4	14.9	17.9
7	777	5.0	7.0	13.0	17.0	20.0
8	774	10.5	14.0	18.0	21.0	24.0
9	771	15.0	17.5	21.0	24.0	27.0
10	767.5	20.0	21.5	24.5	27.5	30.5
11	763	24.5	26.0	29.0	32.0	35.0
12	759	28.5	30.0	33.0	36.0	39.0
13	755	32.5	34.0	37.0	40.0	43.0
14	753	34.5	36.0	39.0	42.0	45.0
15	752	35.5	37.0	40.0	43.0	46.0
16	750.5	37.5	38.5	41.5	44.5	47.5

Note: Elevations are in feet referred to NGVD. HW = headwater, TW = tailwater.

Table 6

Pressures on Spillway Crest

Pressure in Prototype Feet of Water

Crest Length 460 ft; Approach Depth 28 ft

Piezometer No.	El	HW = 783.5 TW = 766.7	HW = 786.5 TW = 772.3	HW = 788.9 TW = 776.8	HW = 790.7 TW = 780.8	HW = 792.6 TW = 784.5
1	768	16.5	19.3	21.8	23.3	25.0
2	773	11.5	14.3	16.3	17.8	19.5
3	776	6.8	6.8	6.0	5.0	4.0
4	777.8	4.0	3.5	2.2	1.0	-0.8
5	778	3.5	3.0	2.0	1.0	0.0
6	777.6	3.4	3.4	2.9	2.4	2.4
7	777	2.8	3.0	2.5	2.0	2.5
8	774	1.8	2.0	1.8	3.0	5.5
9	771	1.5	1.5	2.5	5.5	9.5
10	767.5	1.5	2.8	5.5	10.0	15.5
11	763	3.0	7.0	11.0	17.0	22.5
12	759	8.0	12.5	18.5	24.0	26.8
13	755	13.0	20.0	25.8	29.0	30.8
14	753	17.5	24.3	28.0	31.0	32.8
15	752	18.0	25.5	29.8	32.5	34.0
16	750.5	20.5	28.5	32.5	35.5	36.5

Piezometer No.	El	HW = 794.7 TW = 787.4	HW = 796.2 TW = 789.7	HW = 798.8 TW = 793.7	HW = 800.7 TW = 796.4	HW = 803.0 TW = 799.2
1	768	27.0	29.0	31.0	32.8	35.0
2	773	21.0	23.0	25.0	27.0	29.0
3	776	2.0	1.0	5.0	7.0	12.0
4	777.8	-2.3	-3.8	1.7	6.2	11.2
5	778	-1.5	-2.5	3.5	7.0	13.0
6	777.6	2.2	2.2	7.4	12.4	17.4
7	777	2.8	3.0	10.0	15.0	19.5
8	774	7.5	11.0	16.0	20.0	23.5
9	771	12.5	16.5	20.5	23.3	26.5
10	767.5	19.5	21.5	23.5	27.3	30.0
11	763	24.0	25.0	28.5	31.3	34.5
12	759	28.0	29.0	32.5	35.3	38.5
13	755	32.0	33.0	36.5	39.3	42.5
14	753	34.0	35.0	38.5	41.3	44.5
15	752	35.0	36.0	39.5	42.3	45.5
16	750.5	37.5	38.5	41.3	43.8	47.0

Note: Elevations are in feet referred to NGVD. HW = headwater, TW = tailwater.

Table 7

Pressures on Spillway Crest

Pressure in Prototype Feet of Water

Crest Length 460 ft; Approach Depth 12 ft

Piezometer No.	Piezometer El	HW = 783.6 TW = 766.7	HW = 786.7 TW = 772.3	HW = 789.0 TW = 776.8	HW = 791.3 TW = 780.8	HW = 793.5 TW = 784.5
1	768	16.3	19.5	22.0	23.5	25.5
2	773	11.3	14.5	16.5	18.0	20.0
3	776	6.5	7.0	6.0	5.0	3.8
4	777.8	3.7	3.2	2.2	3.7	-1.0
5	778	3.3	3.0	2.0	3.8	-0.5
6	777.6	3.1	3.4	2.9	2.4	1.9
7	777	2.5	3.0	2.5	1.8	2.0
8	774	1.7	2.0	1.5	1.8	5.0
9	771	1.3	1.5	2.0	3.0	9.0
10	767.5	1.5	2.5	4.5	6.8	15.0
[illegible]	763	3.0	7.0	10.0	14.0	22.5
12	759	8.0	12.5	17.3	22.0	26.8
13	755	13.0	20.0	25.0	27.5	30.8
14	753	17.5	24.5	28.0	30.0	32.8
15	752	18.0	25.8	29.5	31.5	34.0
16	750.5	21.3	28.0	32.0	34.5	36.5

Piezometer No.	Piezometer El	HW = 795.6 TW = 787.4	HW = 798.0 TW = 789.7	HW = 800.2 TW = 793.7	HW = 792.5 TW = 796.4	HW = 804.9 TW = 799.2
1	768	27.0	30.0	31.5	33.5	36.0
2	773	21.5	24.0	25.5	27.5	30.0
3	776	2.5	1.0	3.0	5.0	10.0
4	777.8	-2.3	-3.8	0.2	2.2	9.2
5	778	-1.0	-2.0	2.0	5.0	12.0
6	777.6	2.4	2.4	6.9	10.4	15.4
7	777	3.0	4.0	9.0	13.0	18.0
8	774	8.0	12.0	17.0	19.0	22.0
9	771	15.0	17.5	20.0	22.5	25.5
10	767.5	19.5	21.3	23.5	26.0	29.0
11	763	24.5	25.5	28.0	30.5	33.5
12	759	28.5	29.0	32.0	34.5	37.5
13	755	32.0	33.0	36.0	38.5	41.8
14	753	34.0	35.0	38.0	40.5	43.8
15	752	35.0	36.0	39.0	41.5	44.8
16	750.5	37.5	38.5	40.5	43.0	46.3

Note: Elevations are in feet referred to NGVD. HW = headwater, TW = tailwater.

Table 8

Minimum Pressures (hp) on Upstream Quadrant of Spillway for a 12-ft Design Head (H_d)

HW	TW	H_e	h	hp	$\frac{hp}{H_d}$	$\frac{h}{H_e}$	$\frac{H_e}{H_d}$
790	784.0	12	6.0	+1.7	+0.14	0.50	1.0
	785.2	12	7.2	+2.0	+0.17	0.60	1.0
	786.4	12	8.4	+2.7	+0.23	0.70	1.0
	787.6	12	9.6	+4.0	+0.33	0.80	1.0
	788.8	12	10.8	+8.0	+0.67	0.90	1.0
796.8	787.0	18.8	9.0	-6.8	-0.57	0.48	1.57
	788.8	18.8	10.8	-4.8	-0.40	0.57	1.57
	790.6	18.8	12.6	-1.3	-0.11	0.67	1.57
	792.4	18.8	14.4	+3.2	+0.27	0.77	1.57
	794.2	18.8	16.2	+10.2	+0.85	0.86	1.57
802.9	790.0	24.9	12.0	-16.8	-1.40	0.48	2.08
	792.4	24.9	14.4	-13.8	-1.15	0.58	2.08
	794.8	24.9	16.8	-7.3	-0.61	0.67	2.08
	797.2	24.9	19.2	+2.2	+0.18	0.77	2.08
	799.6	24.9	21.6	+14.2	+1.18	0.87	2.08

Note: Minimum pressure (hp) in feet measured at piezometer No. 4, el 777.8, crest el 778.0, approach depth 28 ft.
HW = headwater.
TW = tailwater.

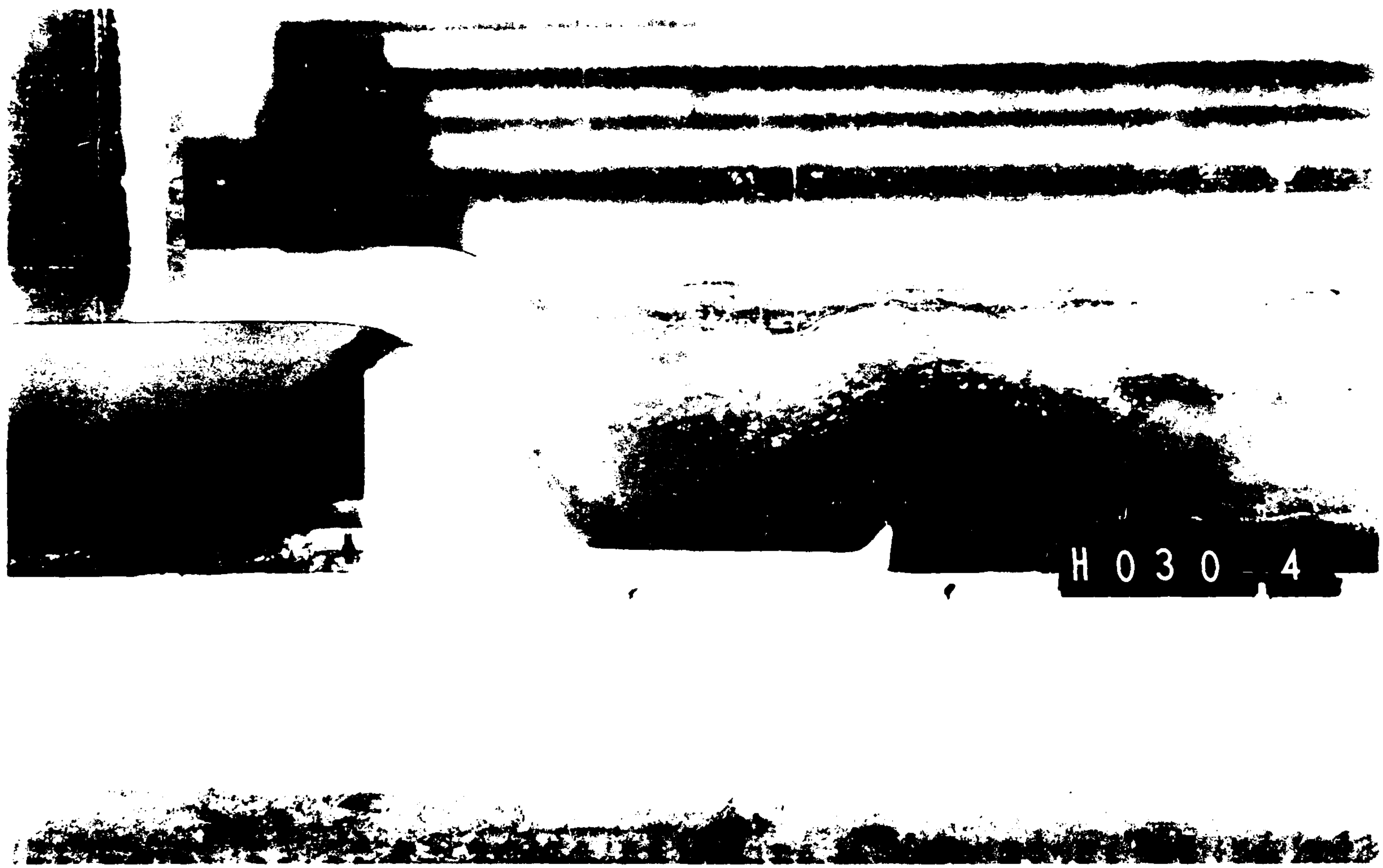


Photo 1. Roller action, plunging nappe, type 1 design; headwater el 784.9, tailwater el 772.3

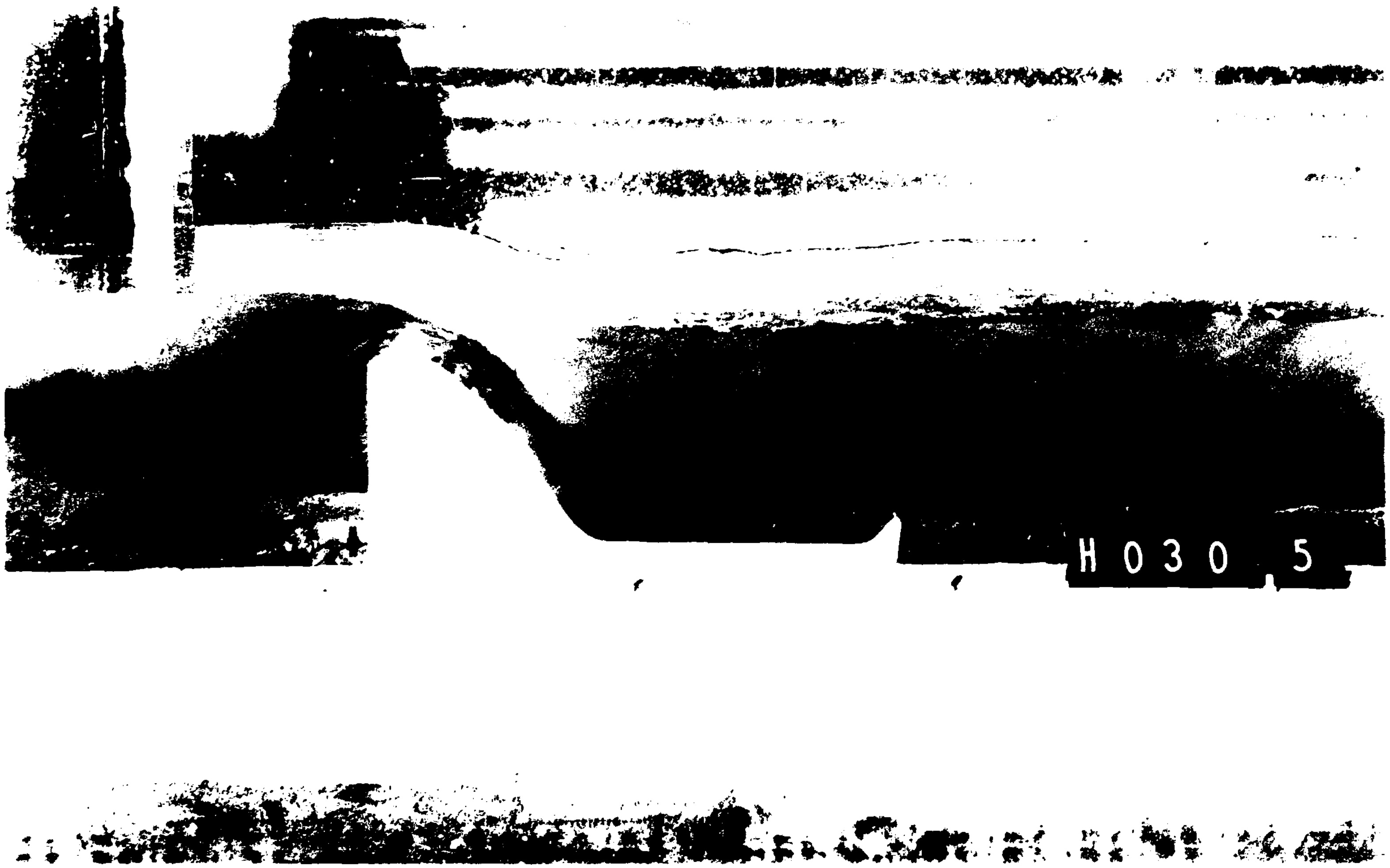


Photo 2. Roller action, plunging nappe, type 1 design; headwater el 789.2, tailwater el 780.8

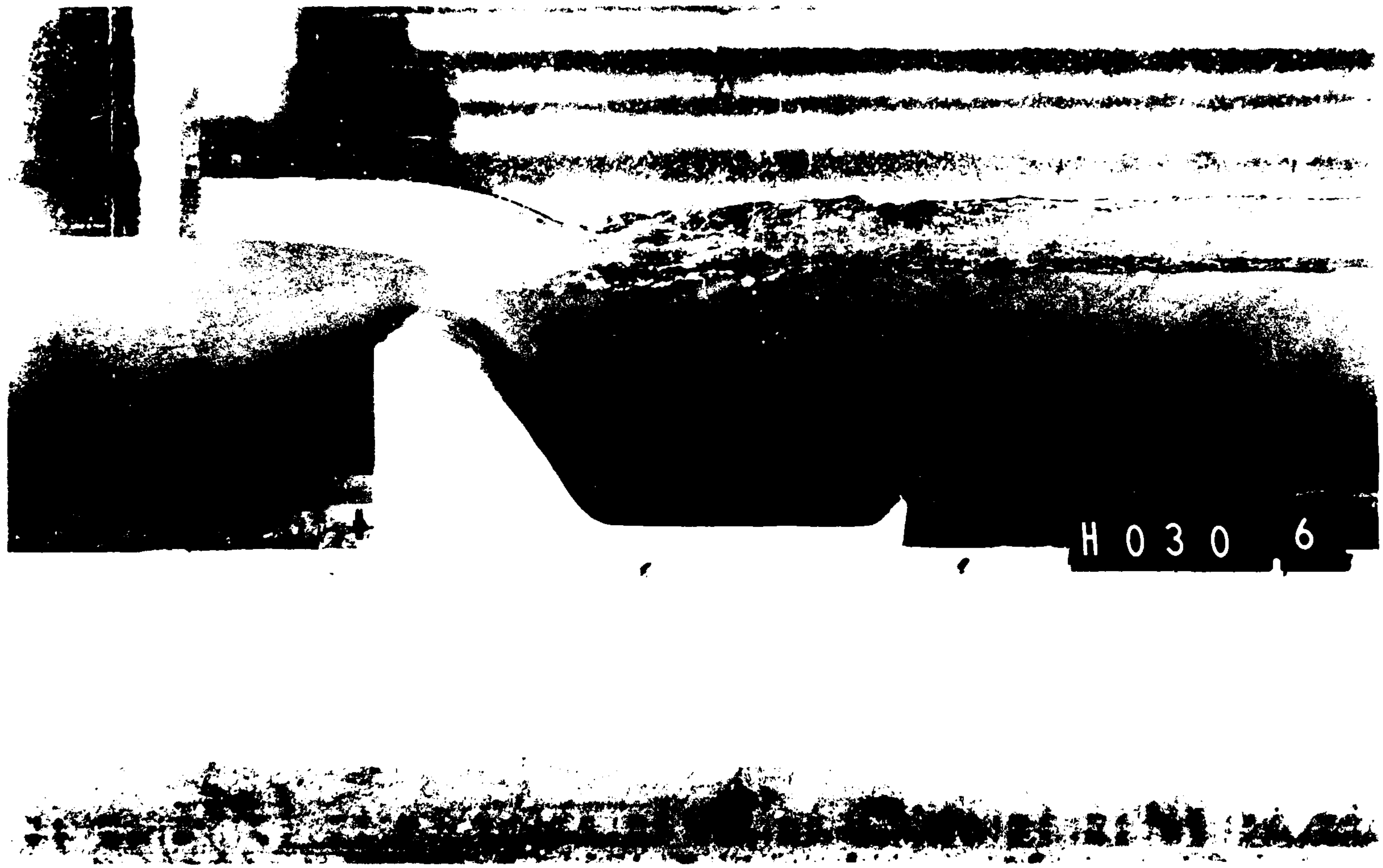


Photo 3. Roller action, plunging nappe, type 1 design; headwater el 794.7, tailwater el 789.7

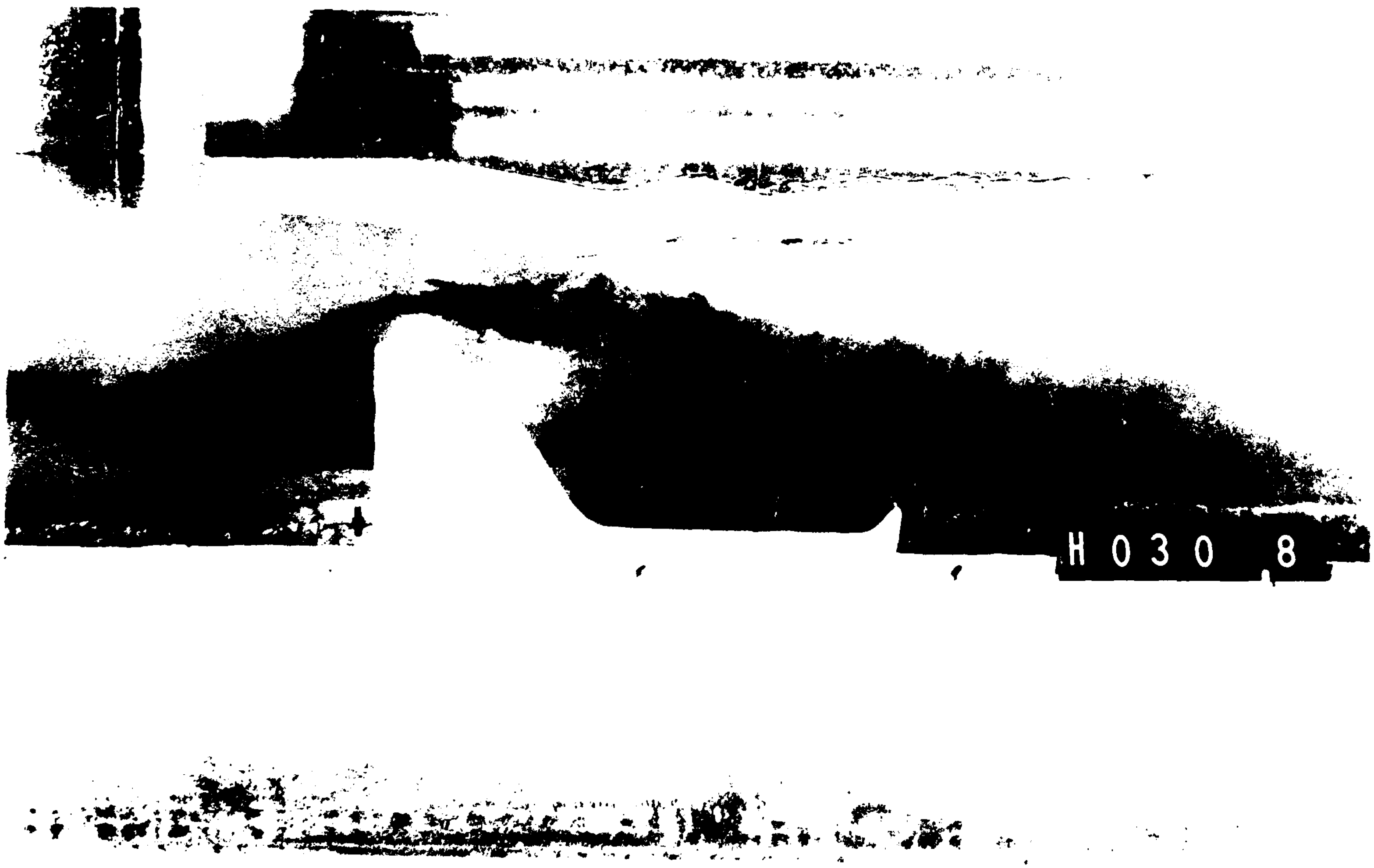


Photo 4. Submerged flow, riding nappe, type 1 design; headwater el 799.5, tailwater el 796.4


BAFFLES

Photo 6. Submerged hydraulic jump, plunging nappe, type 2 design; headwater el 789.2, tailwater el 780.8

Photo 7. Undulating jet, submerged nappe, type 2 design; headwater el 794.7, tailwater el 789.7

Photo 8. Submerged hydraulic jump, plunging nappe, type 3 design; headwater el 784.9, tailwater el 772.3

Photo 9. Submerged hydraulic jump, plunging nappe, type 3 design; headwater el 789.2, tailwater el 780.8

Photo 10. Submerged hydraulic jump, plunging nappe, type 3 design; headwater el 794.7, tailwater el 789.7

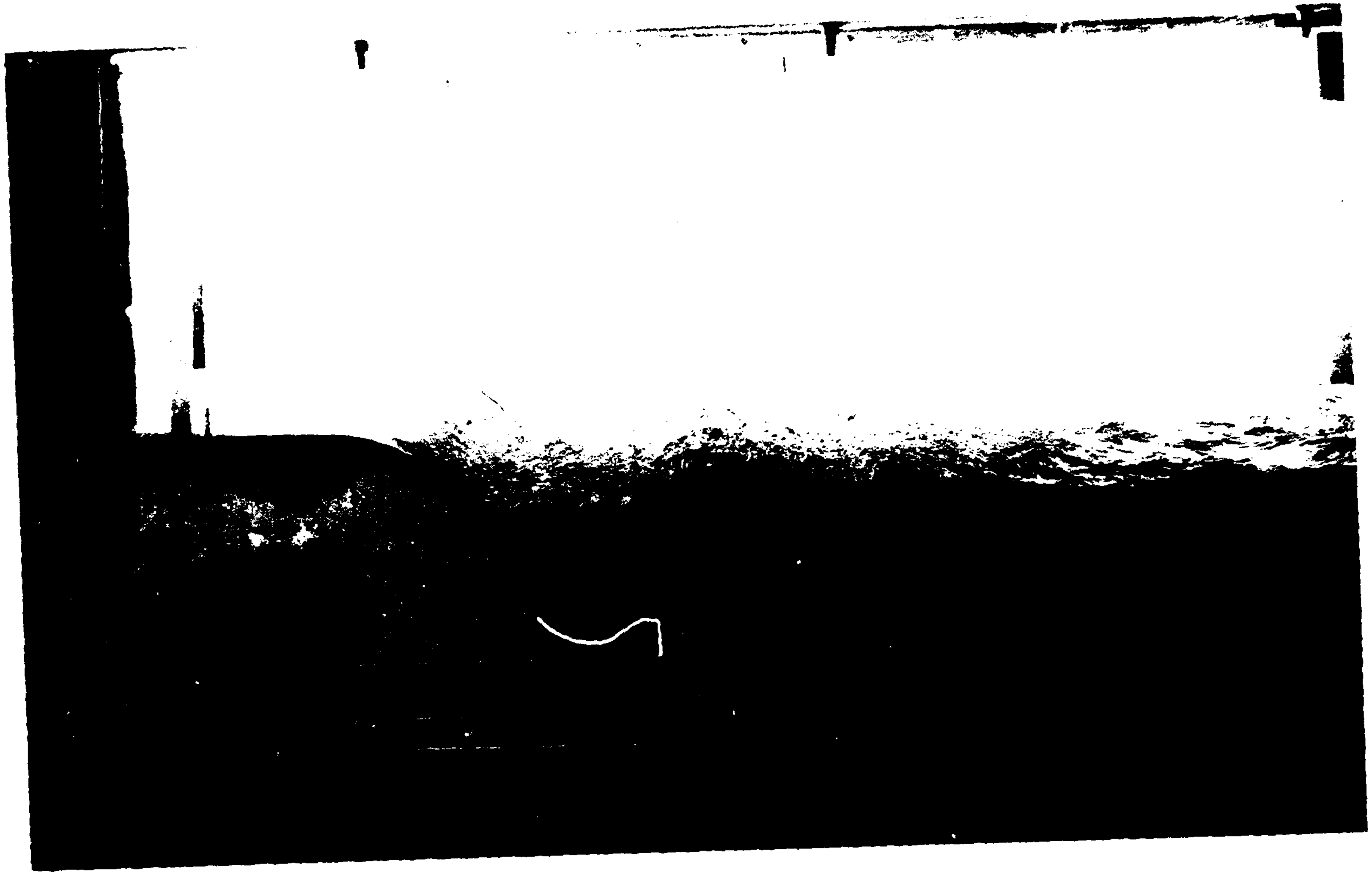


Photo 11. Roller action, surface waves, type 4 design; headwater el 784.9, tailwater el 772.3

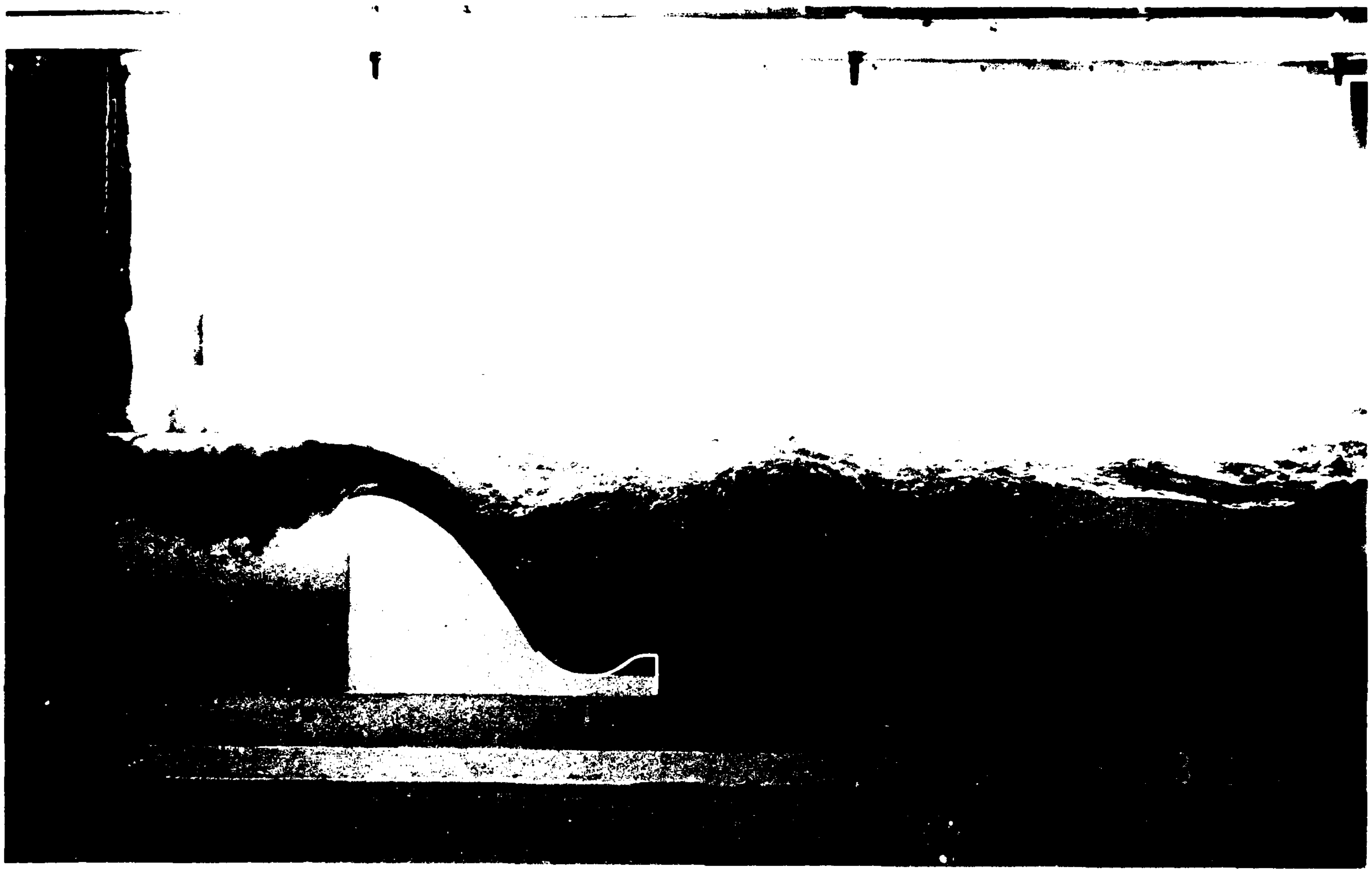


Photo 12. [illegible], surface waves, type 4 design;
[illegible], tailwater el 780.8

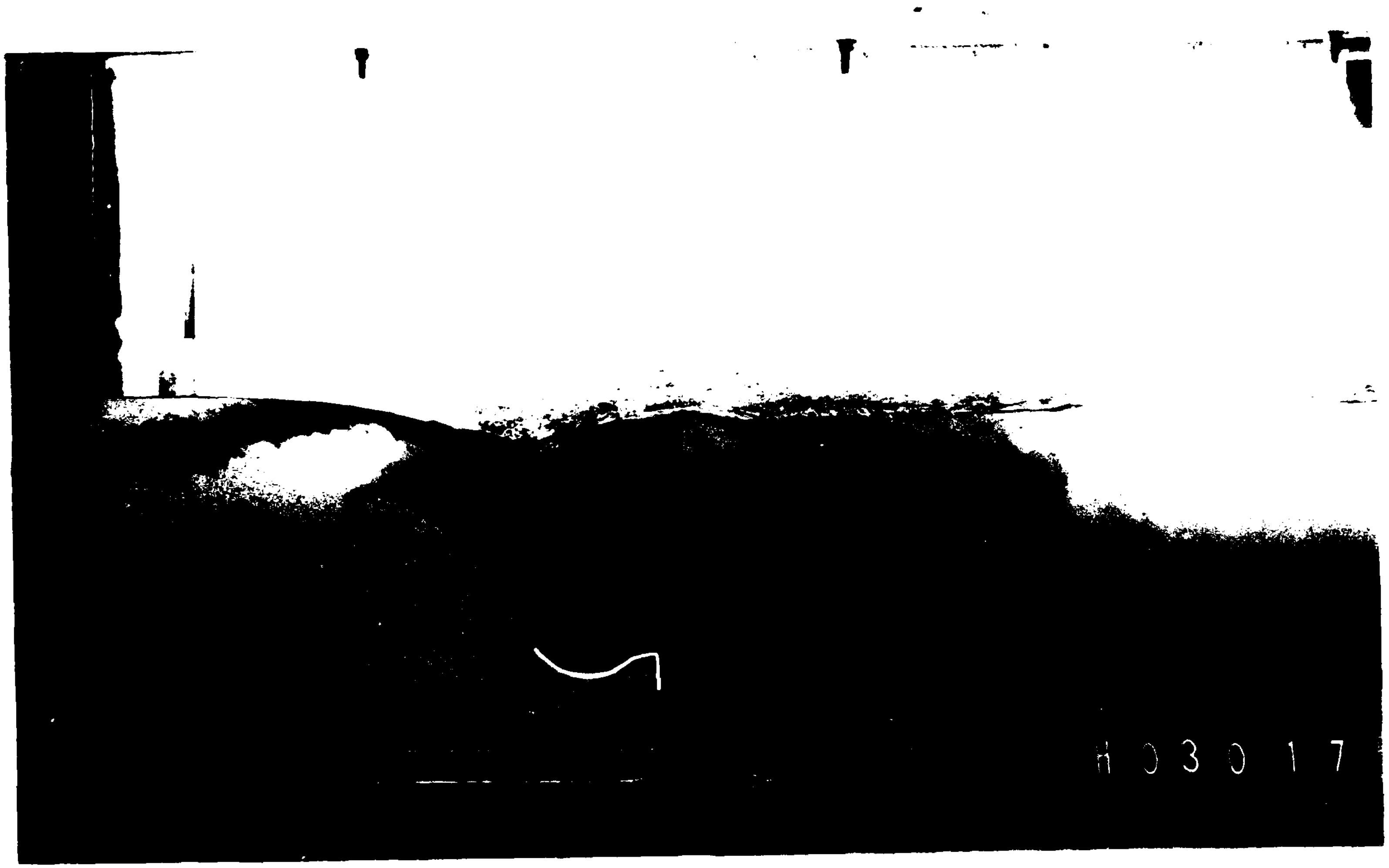


Photo 13. Submerged flow, undulating nappe, type 4 design; headwater el 794.7, tailwater el 789.7

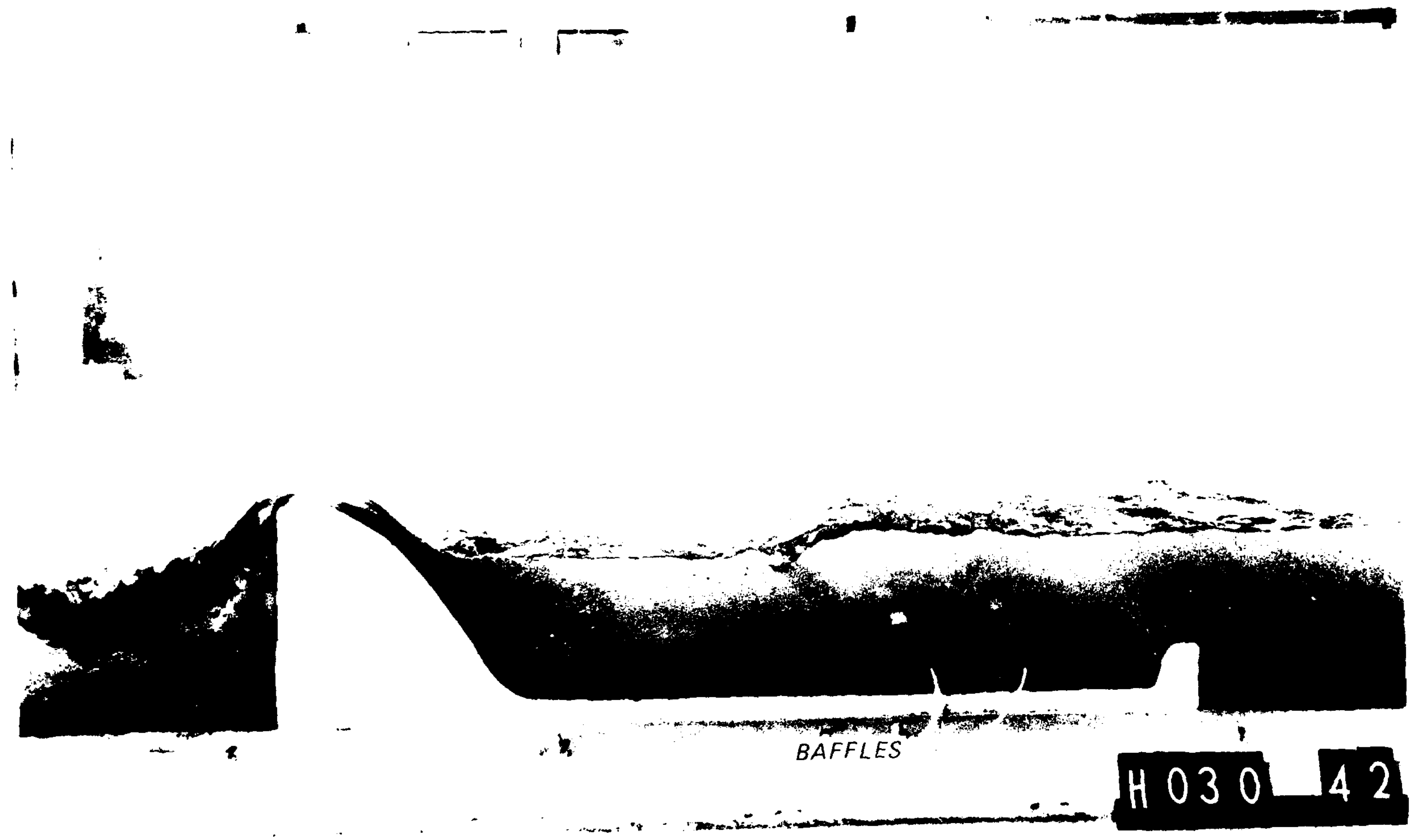


Photo 14. Submerged hydraulic jump, plunging nappe, type 5 design; headwater el 784.9, tailwater el 772.3

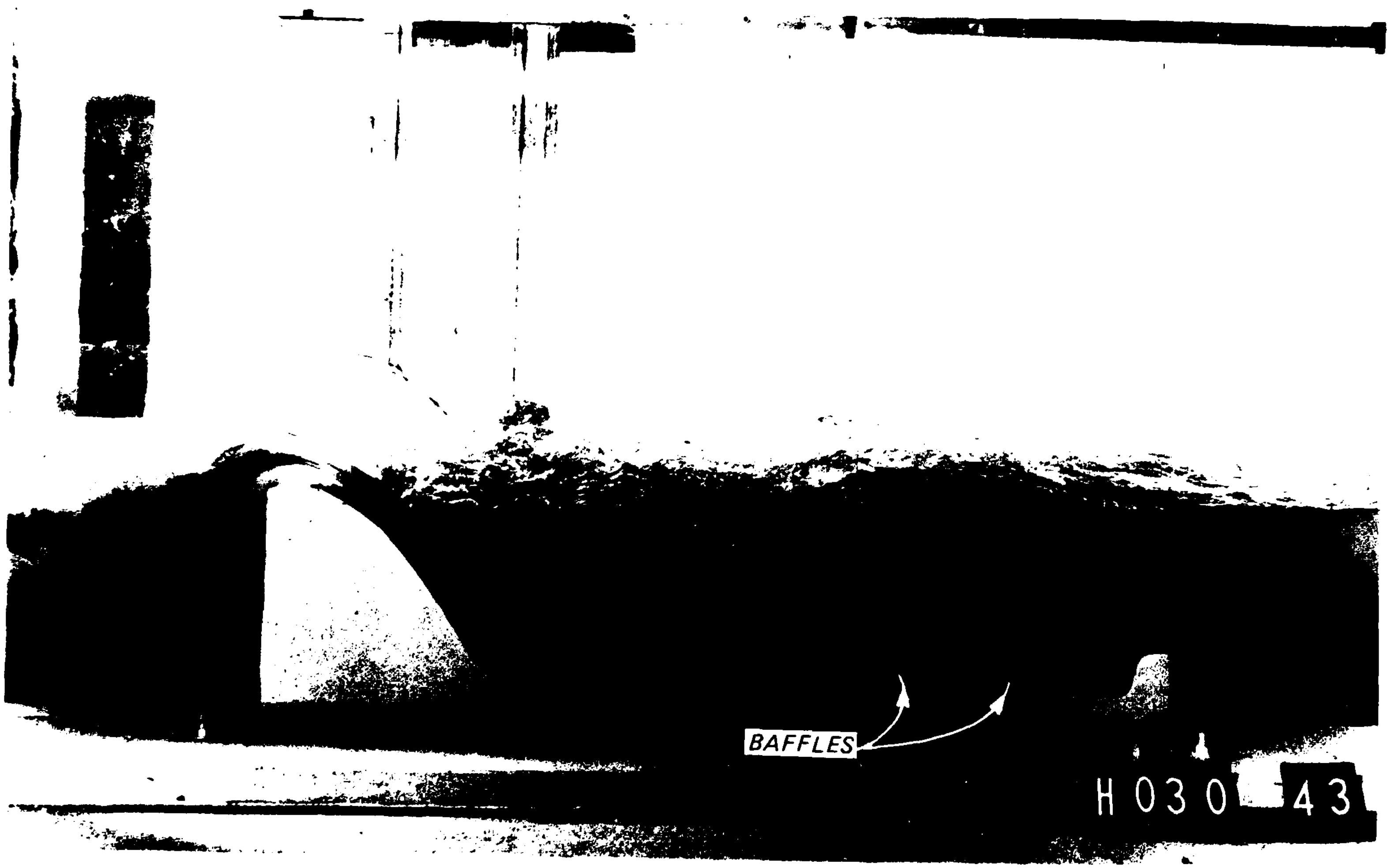


Photo 15. Submerged hydraulic jump, plunging nappe, type 5 design; headwater el 787.2, tailwater el 776.8

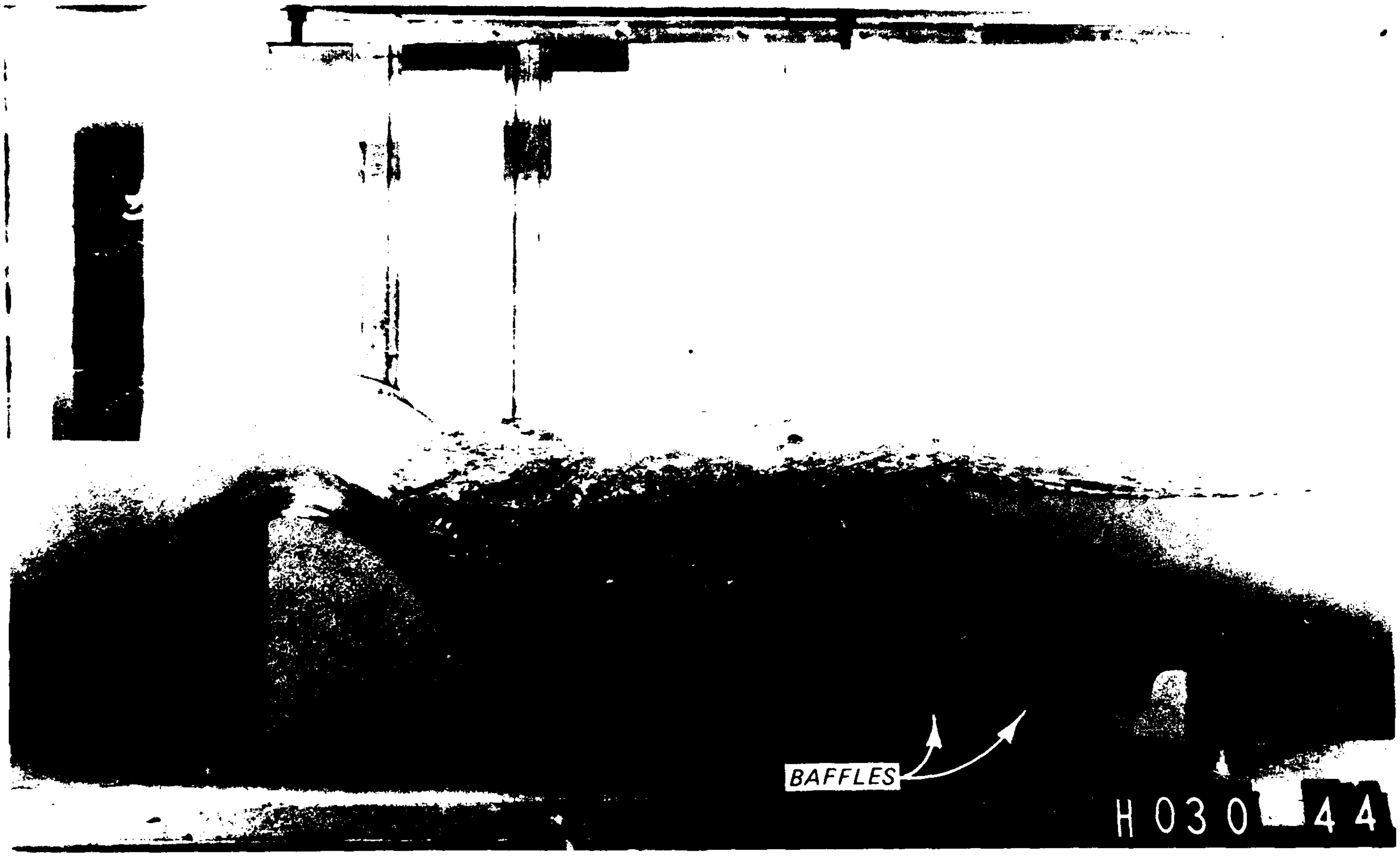


Photo 16. Submerged hydraulic jump, plunging nappe, type 5 design; headwater el 789.2, tailwater el 780.8

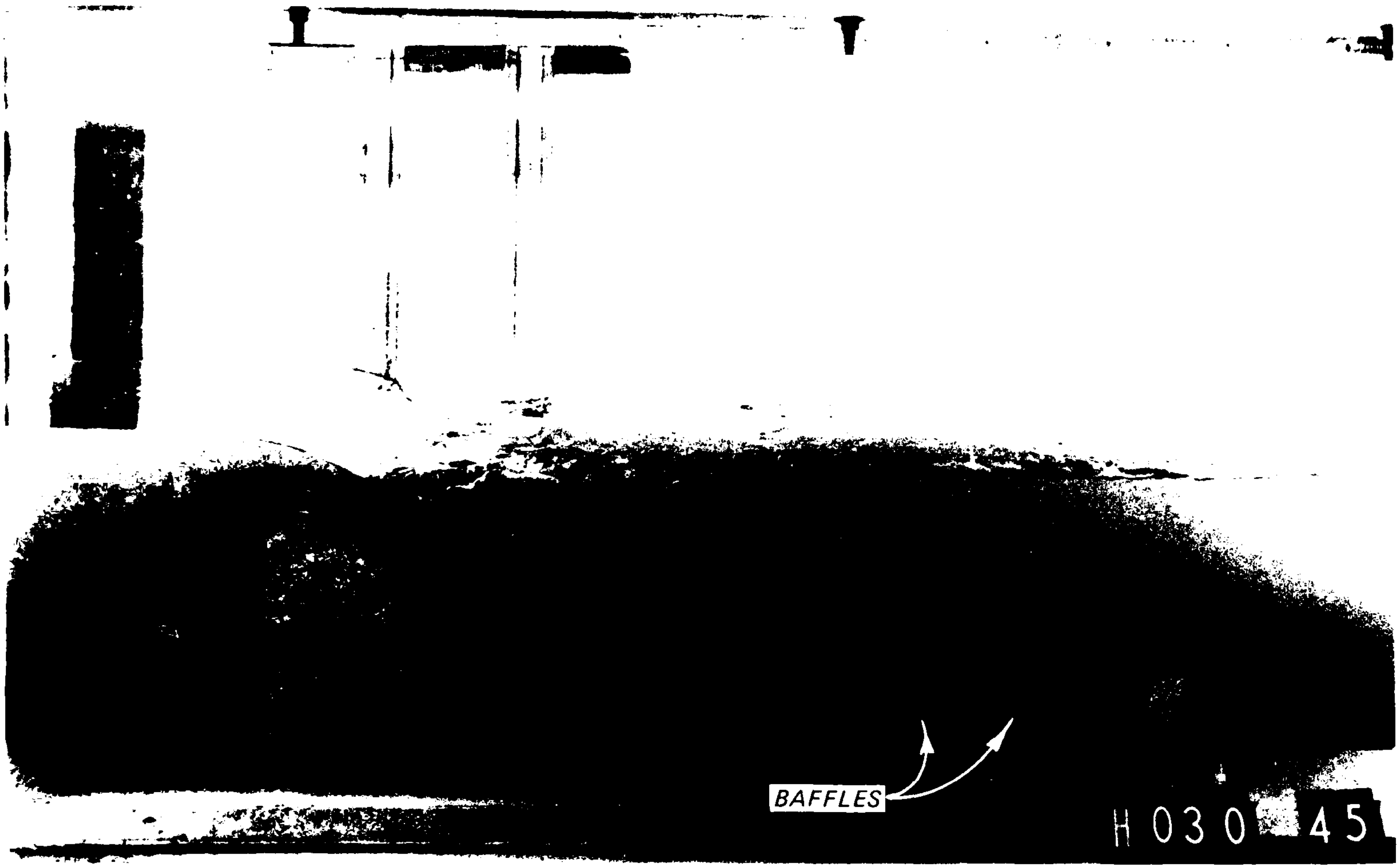


Photo 17. Submerged hydraulic jump, plunging nappe, type 5 design; headwater el 790.7, tailwater el 784.5

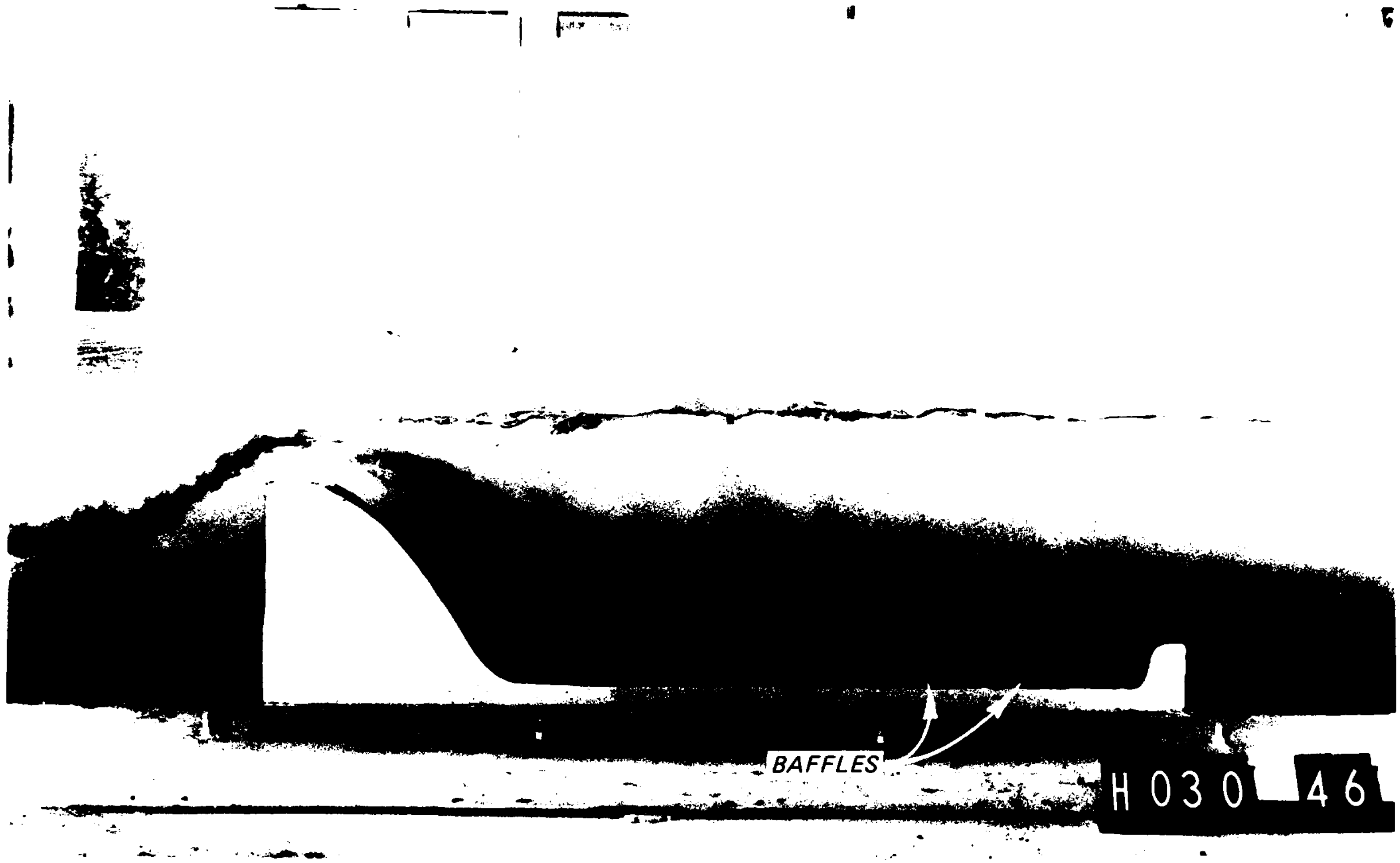


Photo 18. Submerged flow, undulating nappe, type 5 design; headwater el 792.5, tailwater el 787.4

Photo 19. Submerged flow, undulating nappe, type 5 design; headwater el 794.7, tailwater el 789.7

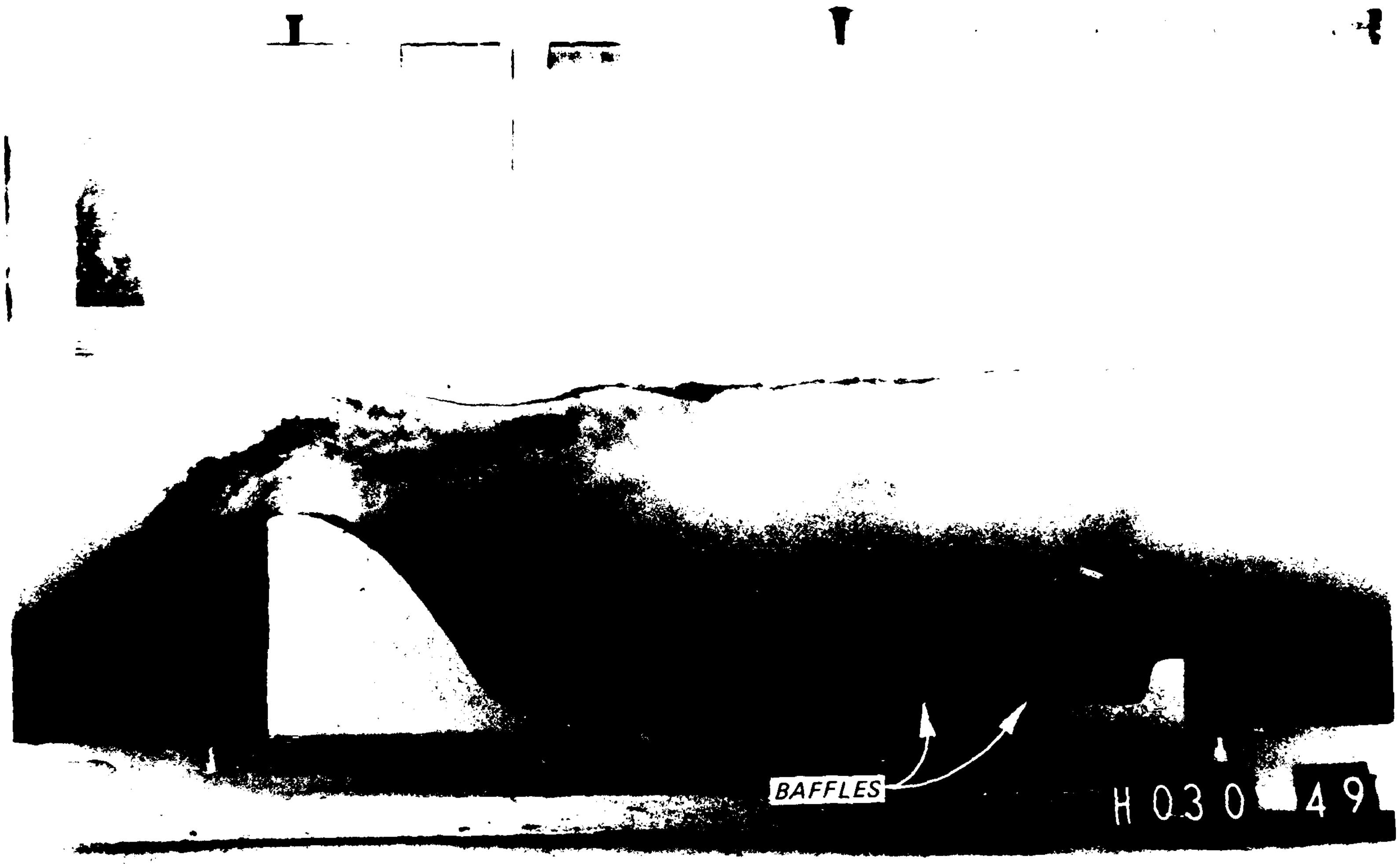


Photo 20. Submerged flow, surface nappe, type 5 design; headwater el 799.5, tailwater el 796.4

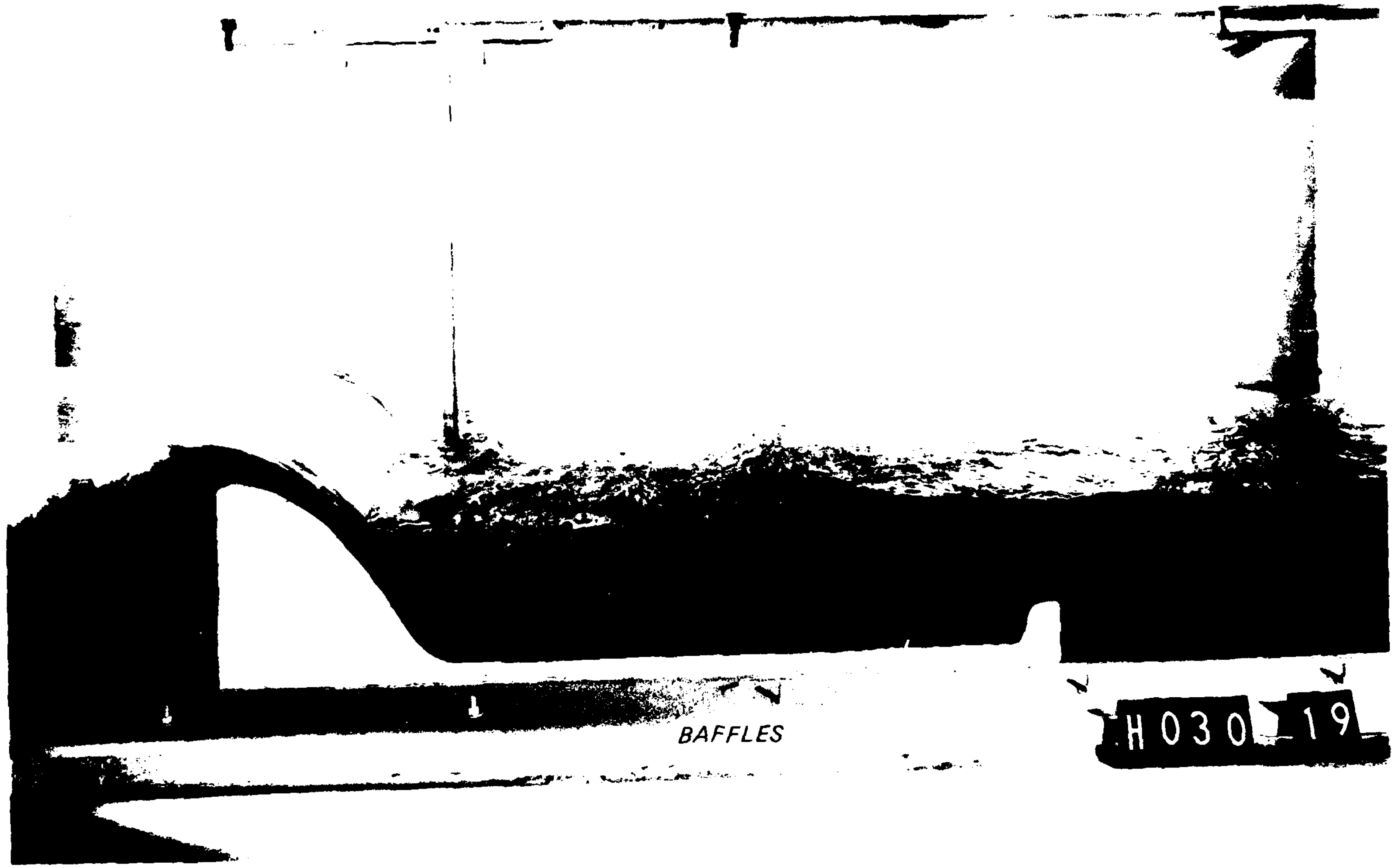


Photo 21. Submerged hydraulic jump, plunging nappe, type 6 design; headwater el 784.9, tailwater el 772.3

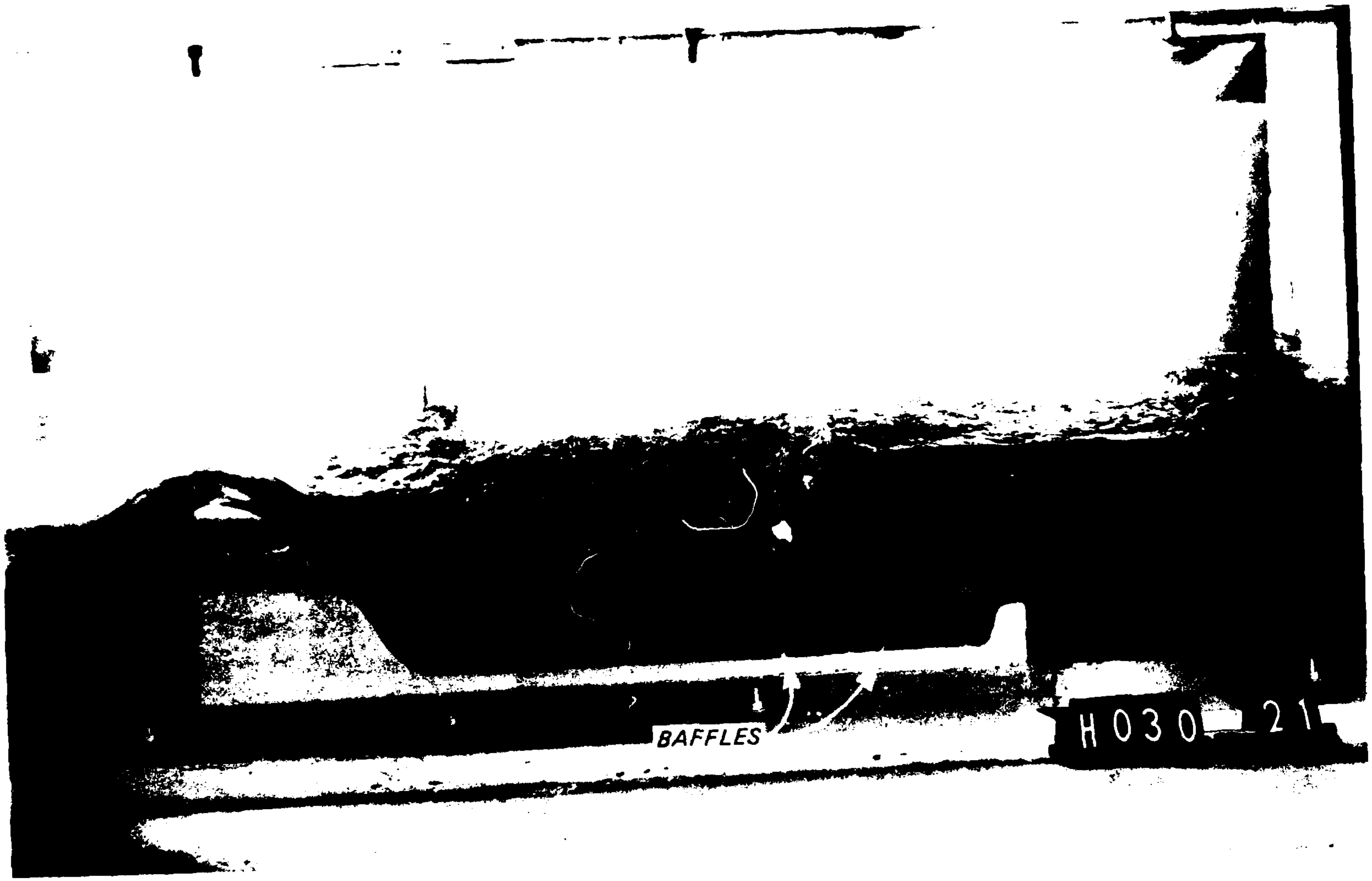


Photo 22. Submerged hydraulic jump, plunging nappe, type 6 design; headwater el 789.2, tailwater el 780.8

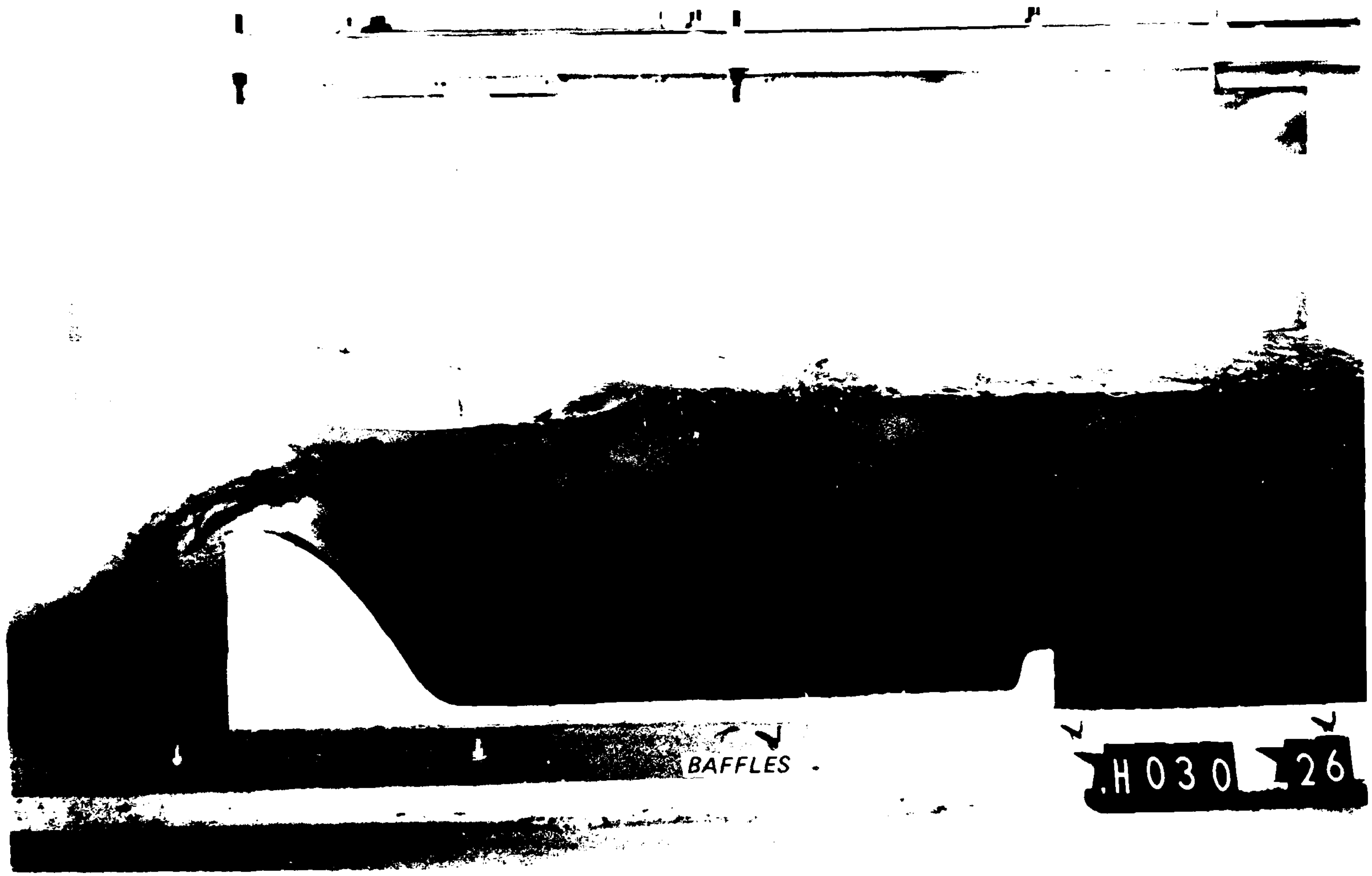


Photo 23. Submerged flow, surface nappe, type 6 design; headwater el 799.5, tailwater el 796.4

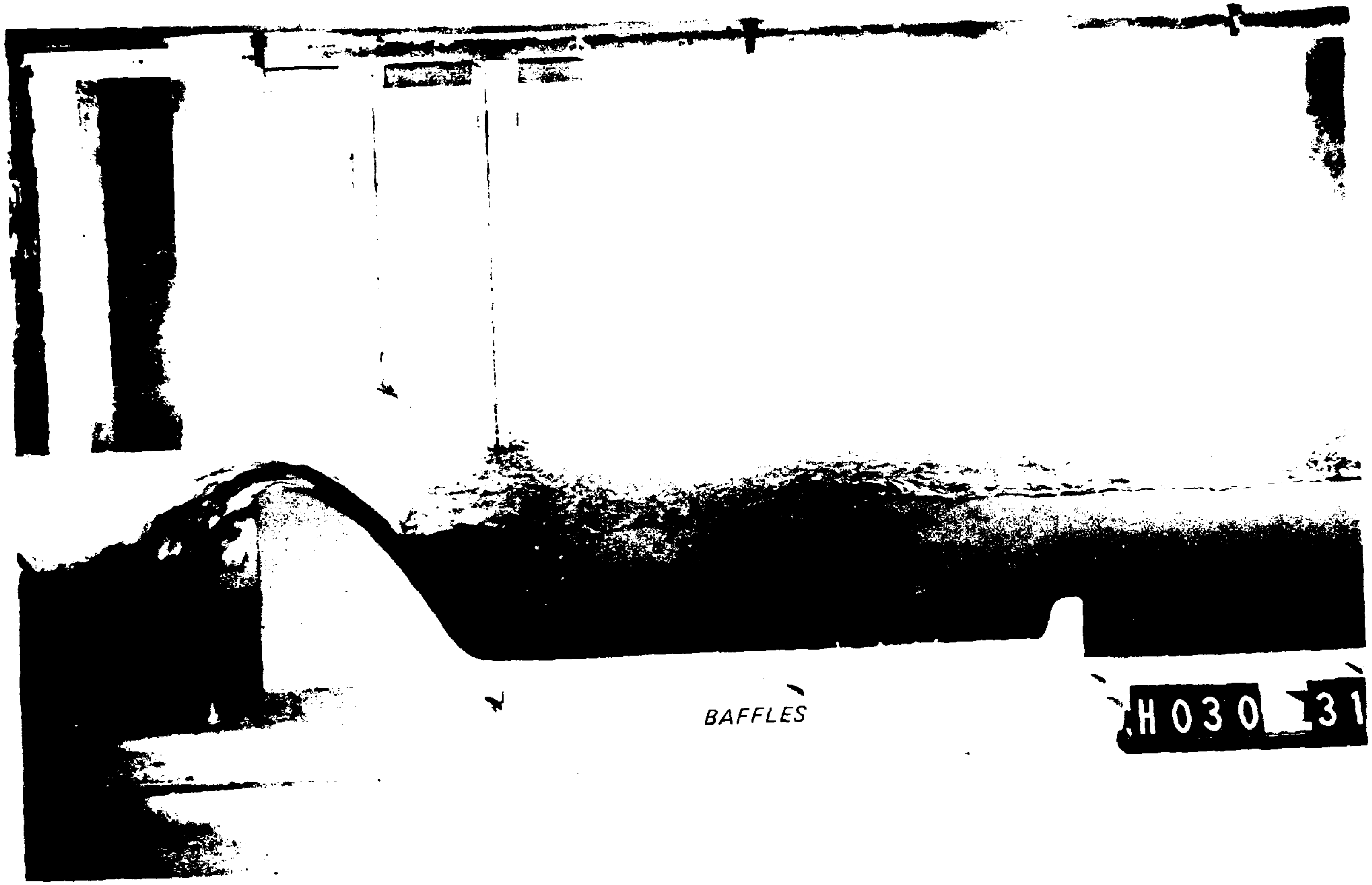


Photo 24. Submerged hydraulic jump, plunging nappe, type 7 design; headwater el 784.9, tailwater el 772.3

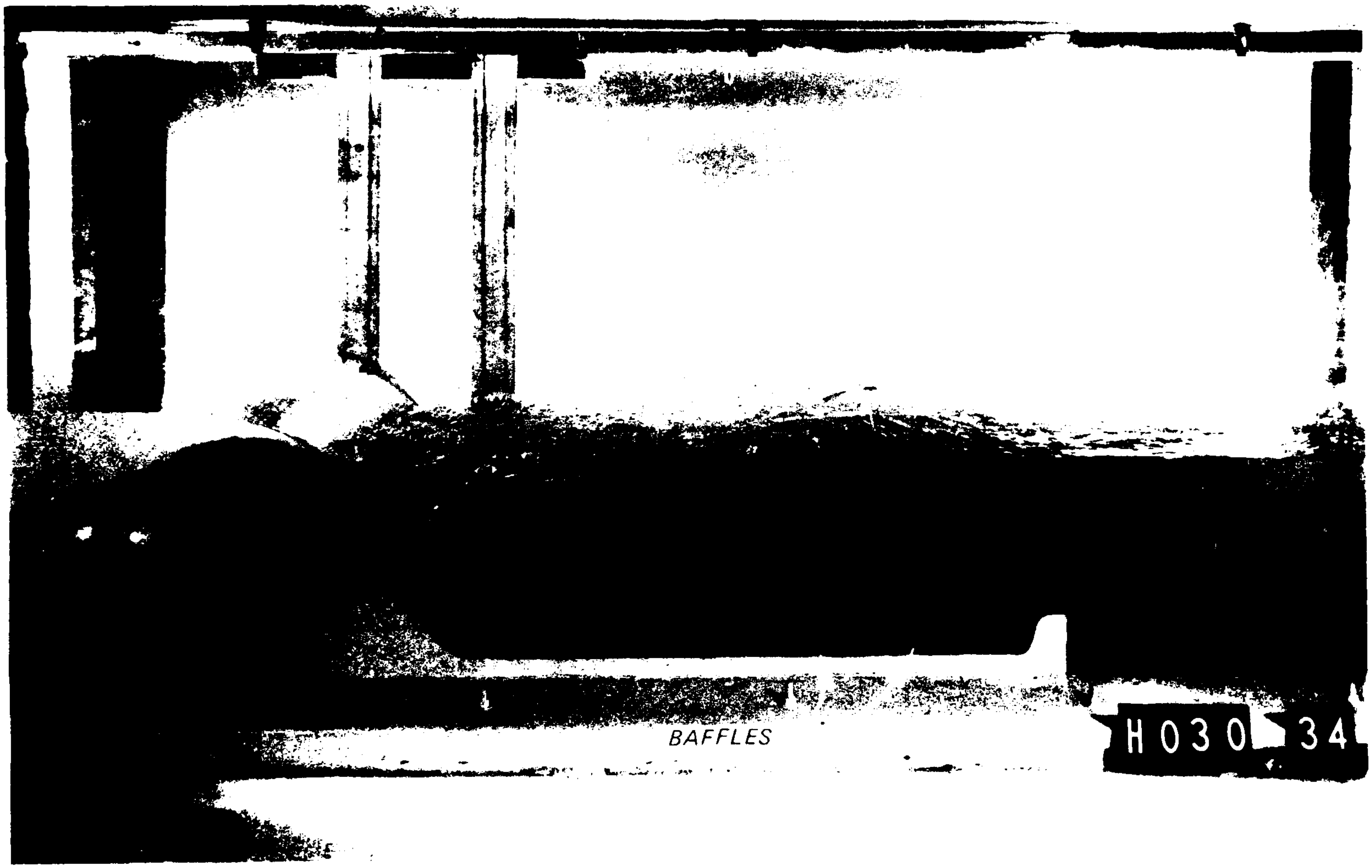


Photo 25. Submerged hydraulic jump, plunging nappe, type 7 design; headwater el [illegible], tailwater el [illegible]

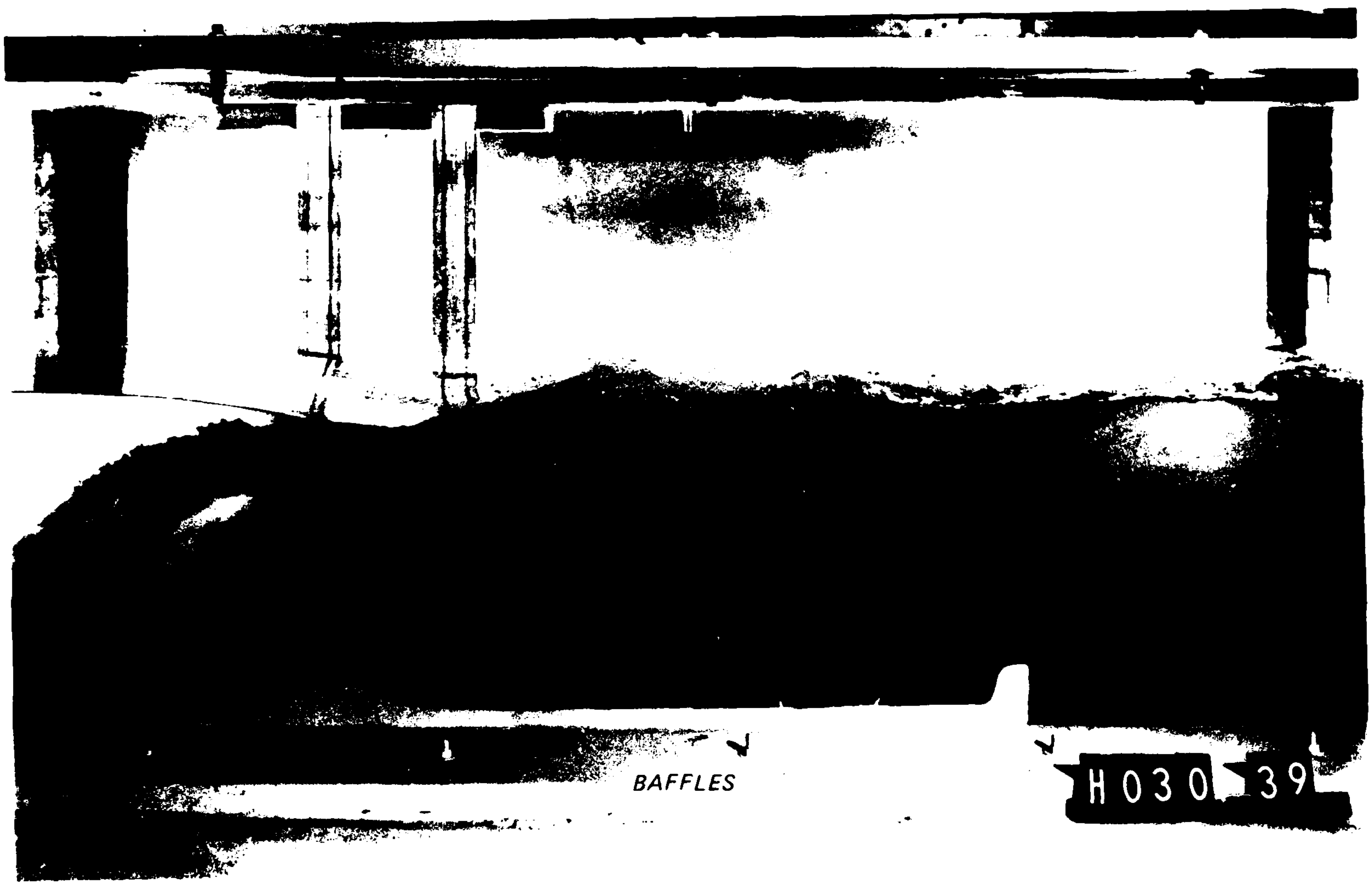


Photo 26. Submerged flow, undulating surface nappe, type 7 design; headwater el 799.5, tailwater el 796.4

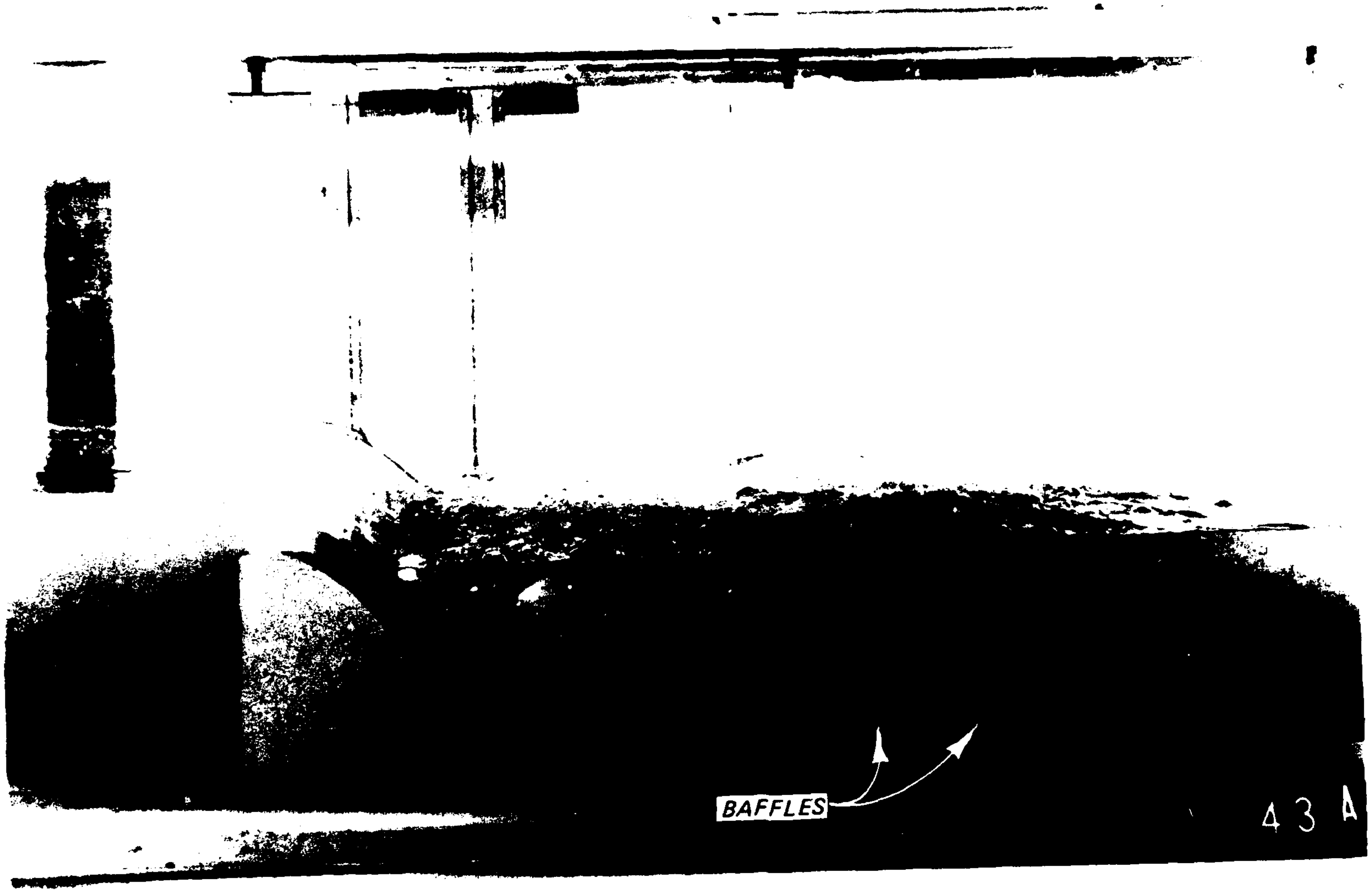


Photo 27. Ice passage test, type 5 design; headwater el 787.2, tailwater el 776.8

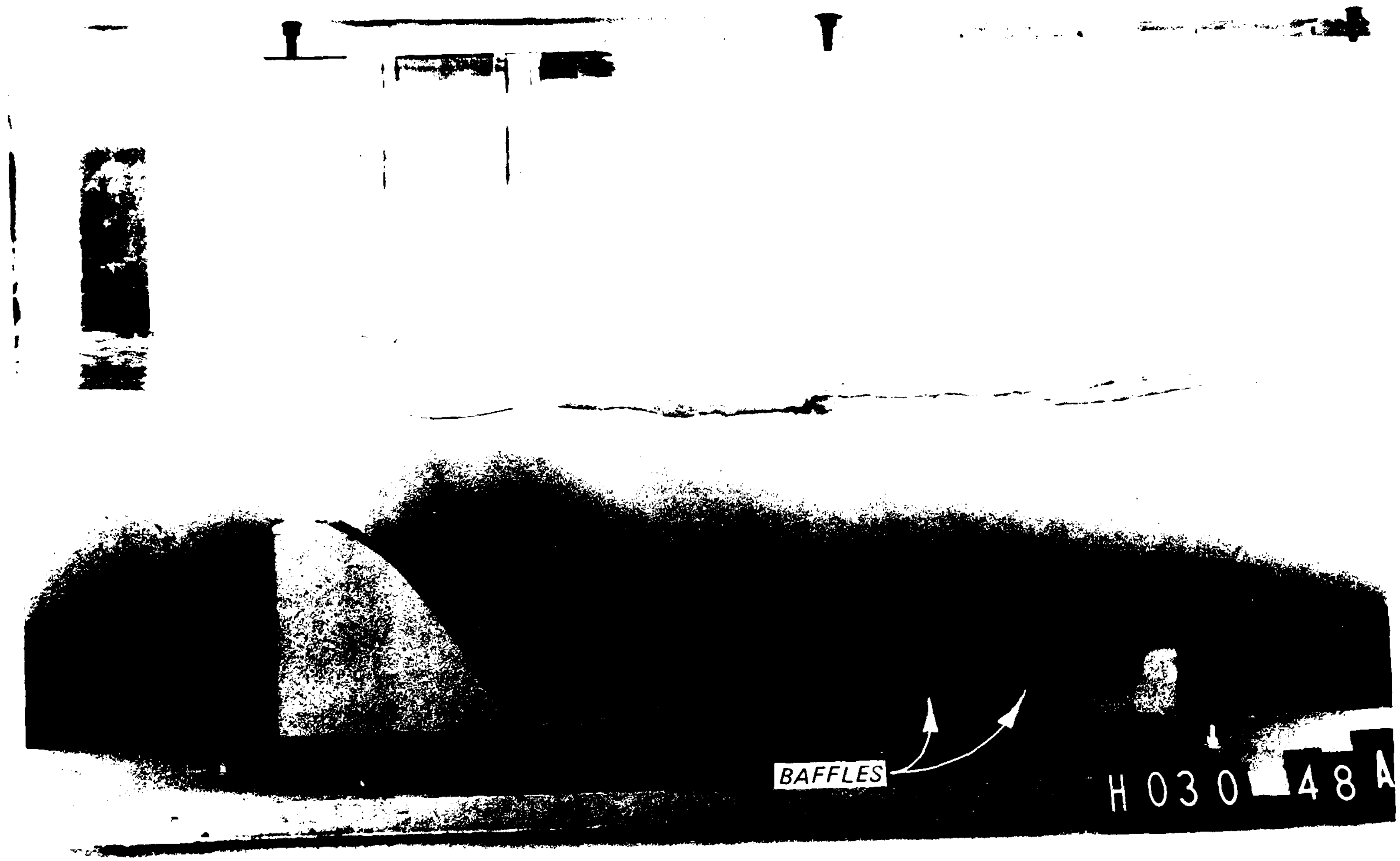


Photo 28. Ice passage test, type 5 design; headwater el 797.5, tailwater el 793.7

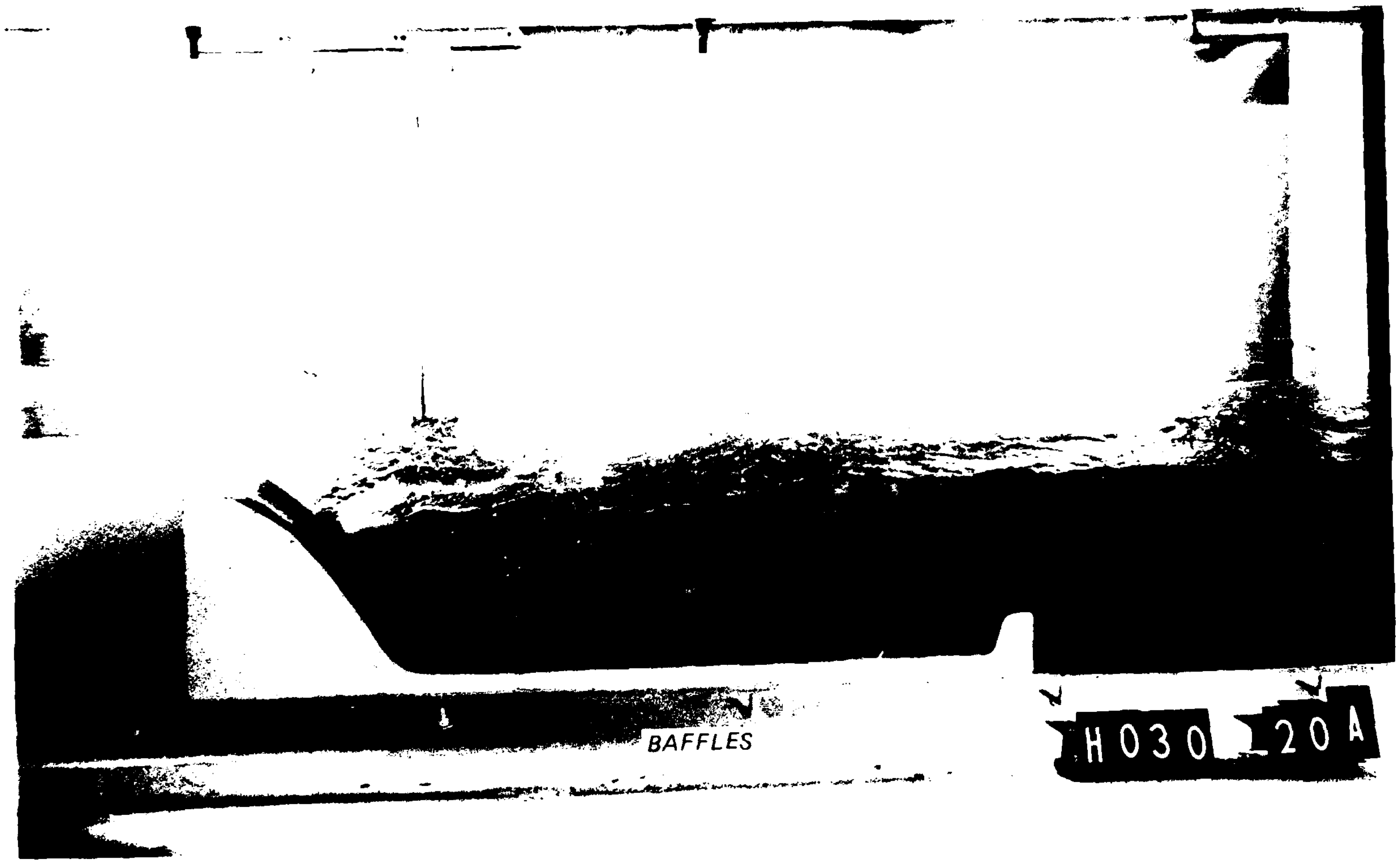


Photo 29. Ice passage test, type 6 design; headwater el 784.9, tailwater el 776.8

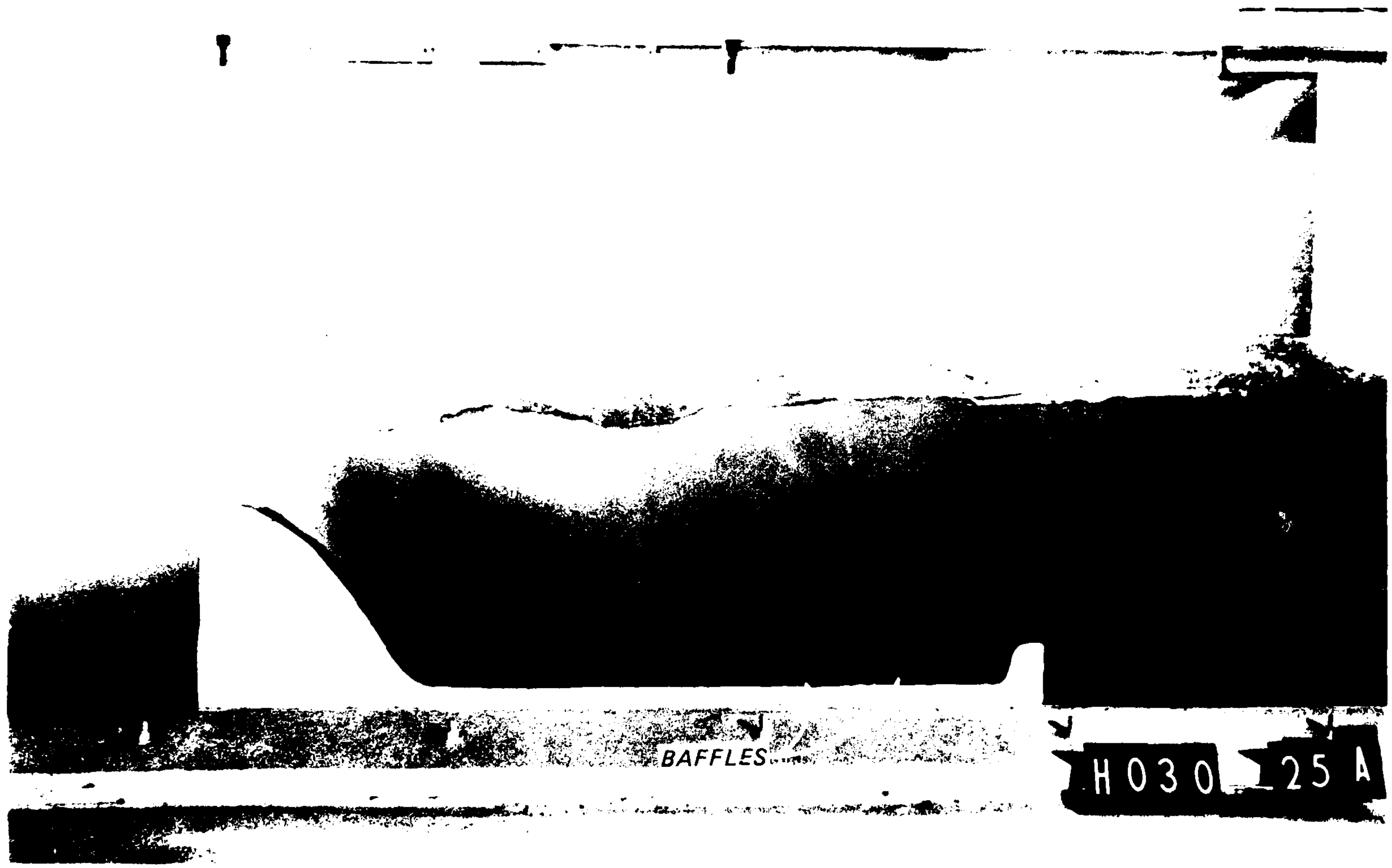


Photo 30. Ice passage test, type 6 design; headwater el 797.5, tailwater el 793.7

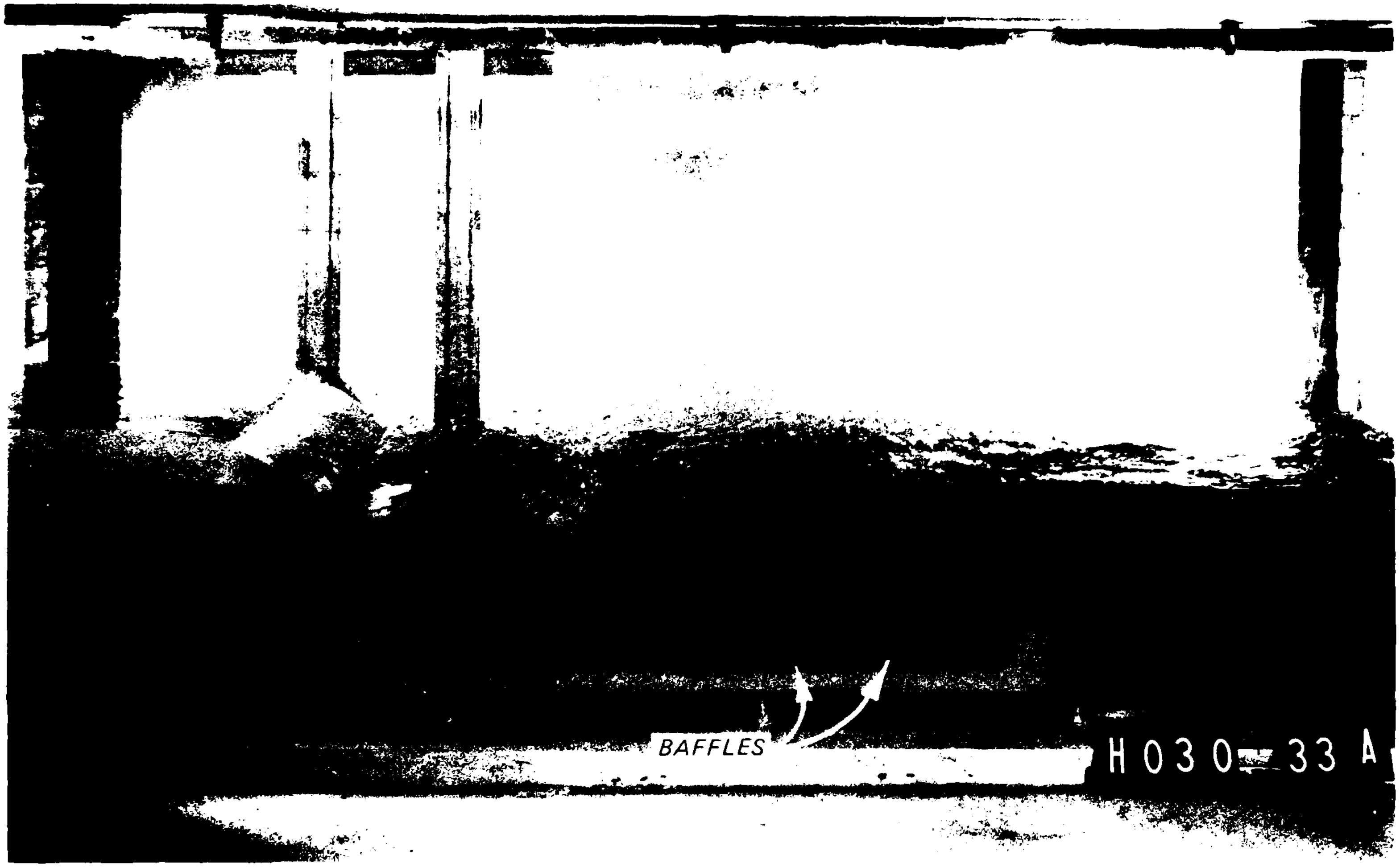


Photo 31. Ice passage test, type 7 design; headwater el 787.2, tailwater el 776.8

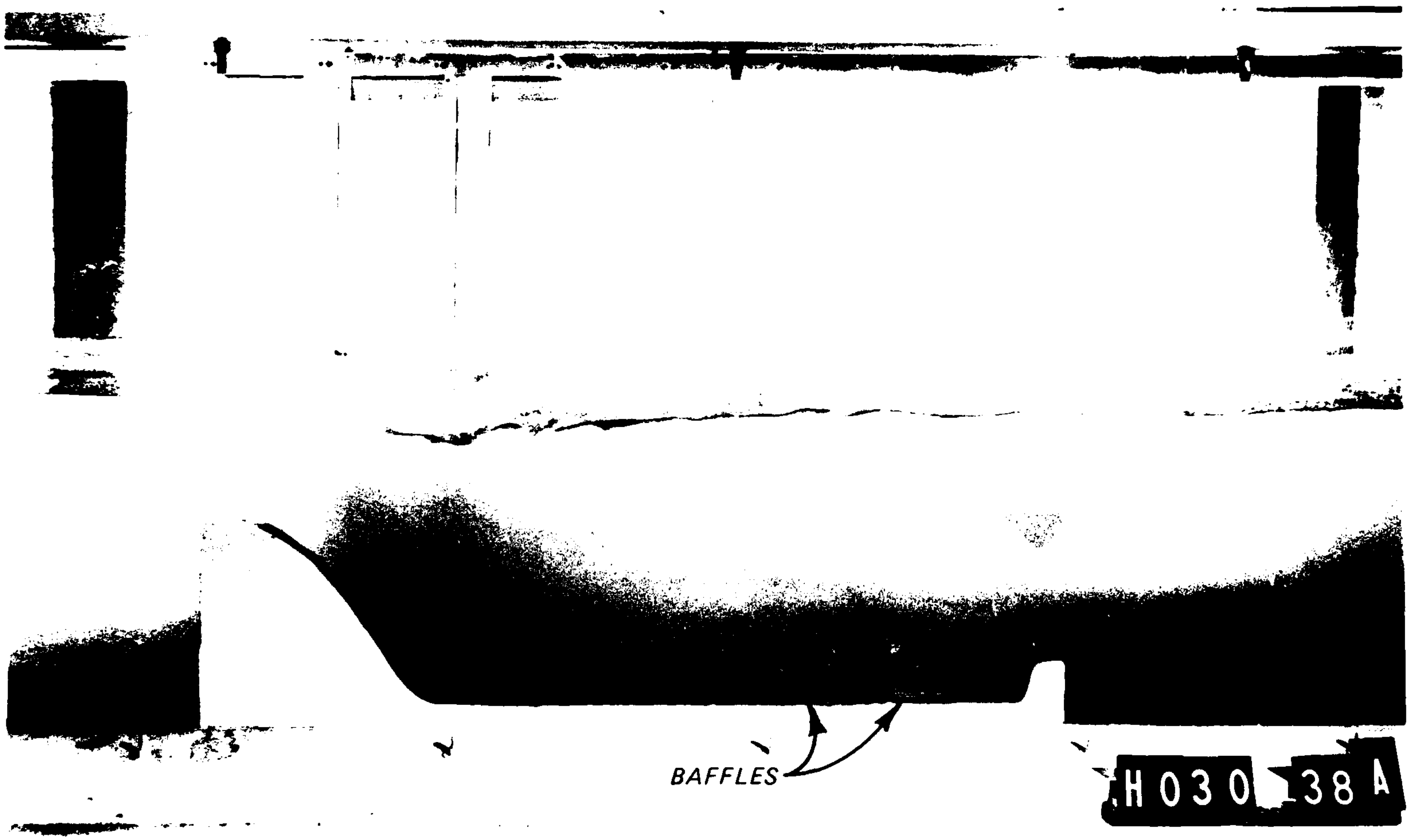
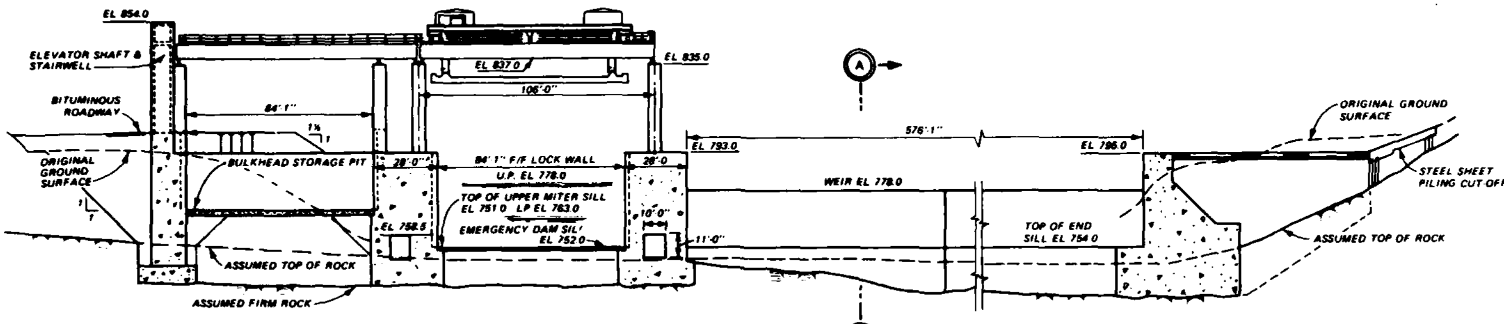


Photo 32. Ice passage test, type 7 design; headwater el 797.5, tailwater el 793.7

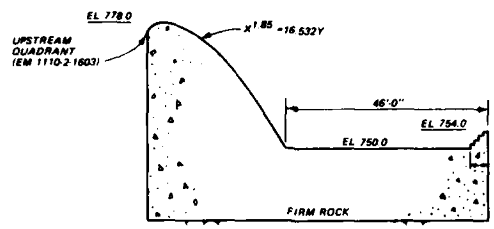


SCALE

20 10 0 20 40FT

1" = 20'

ELEVATION
(LOOKING UPSTREAM)



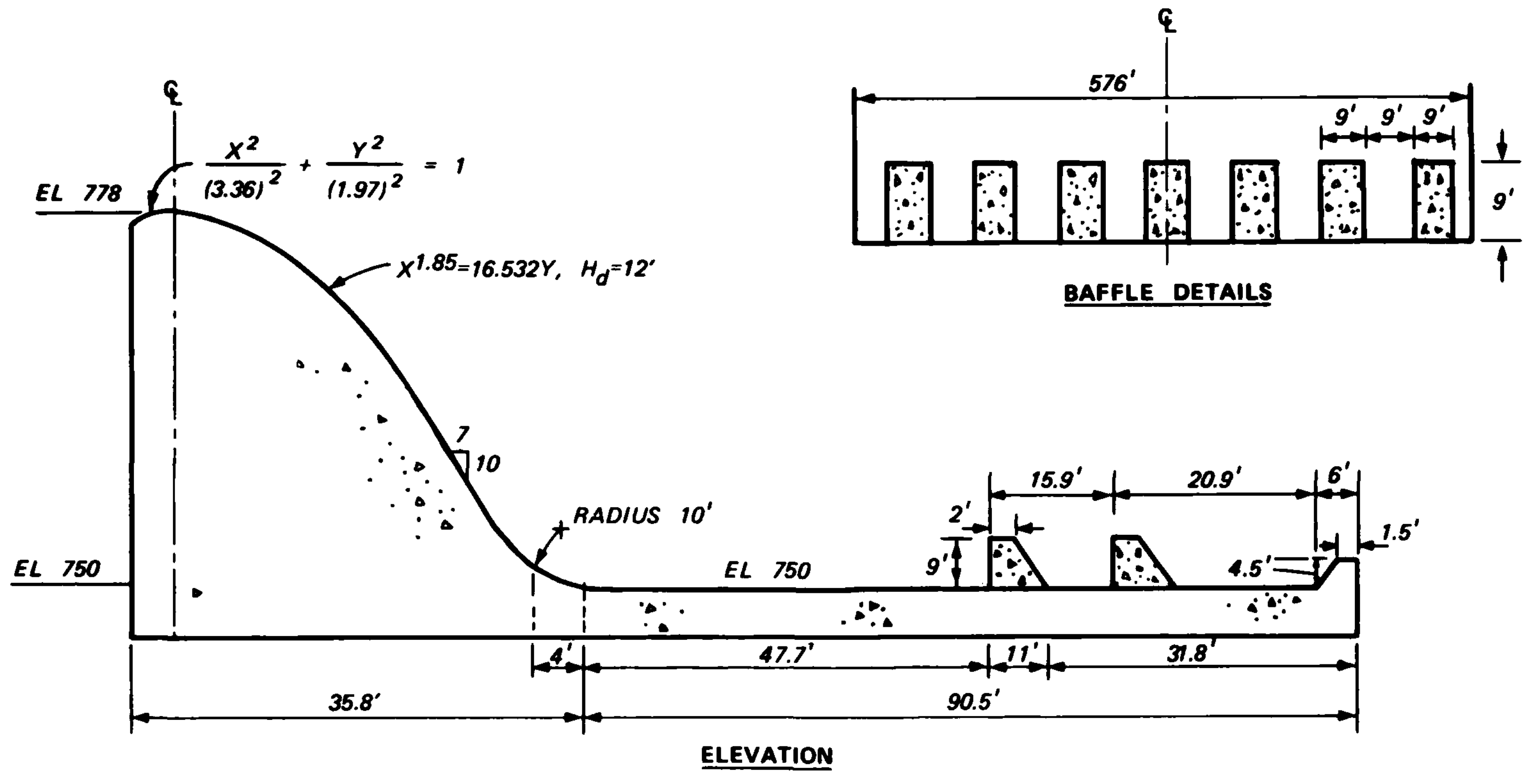
SECTION-A-A

SCALE

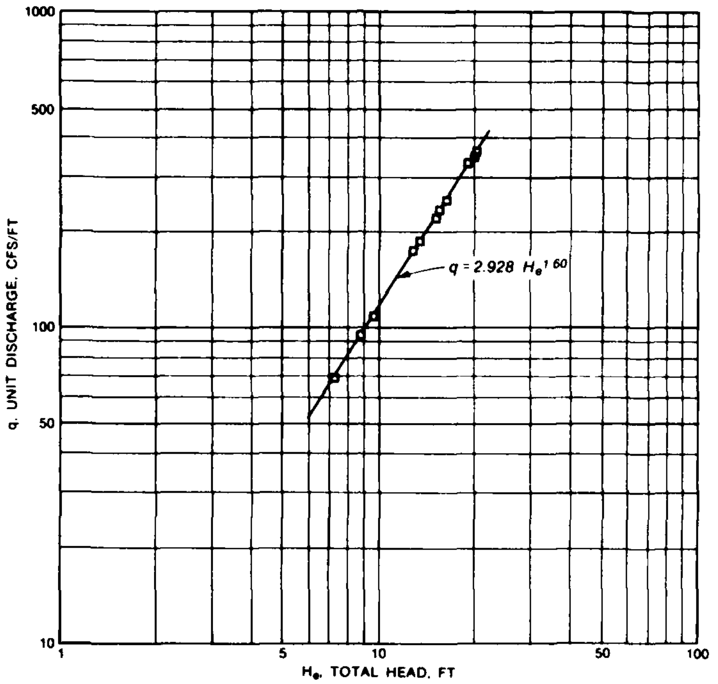
10 5 0 10 20FT

1" = 10'

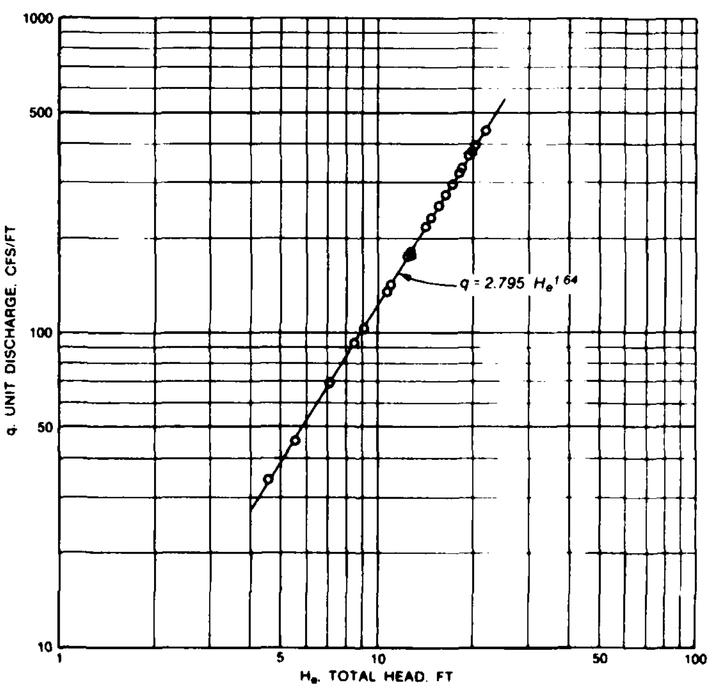
PROJECT DETAILS



TYPE 5 (RECOMMENDED)
DESIGN ENERGY DISSIPATOR



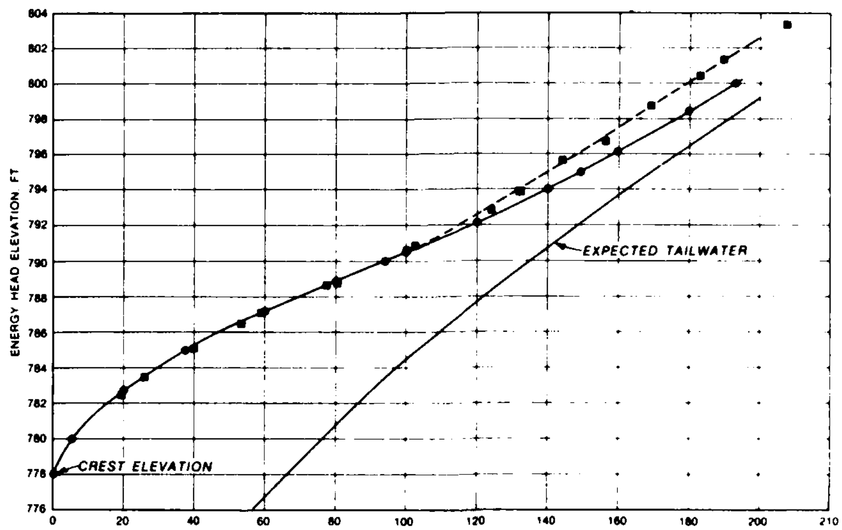
12-FT APPROACH DEPTH



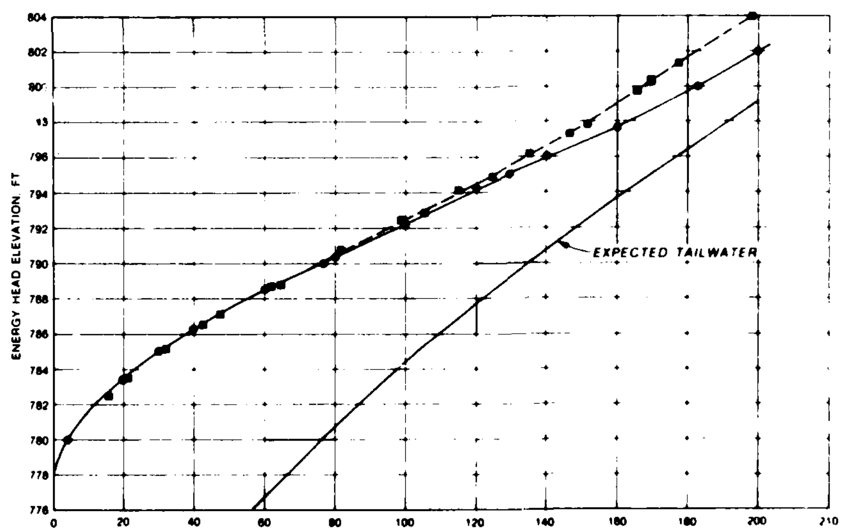
28-FT APPROACH DEPTH

NOTE: H_e = ENERGY HEAD ON CREST, FT
q = UNIT DISCHARGE, CFS/FT

q VS H_e
FREE UNCONTROLLED FLOW
CREST EL 778.0



Q THOUSANDS OF CFS
28-FT APPROACH DEPTH
CREST LENGTH 576 FT

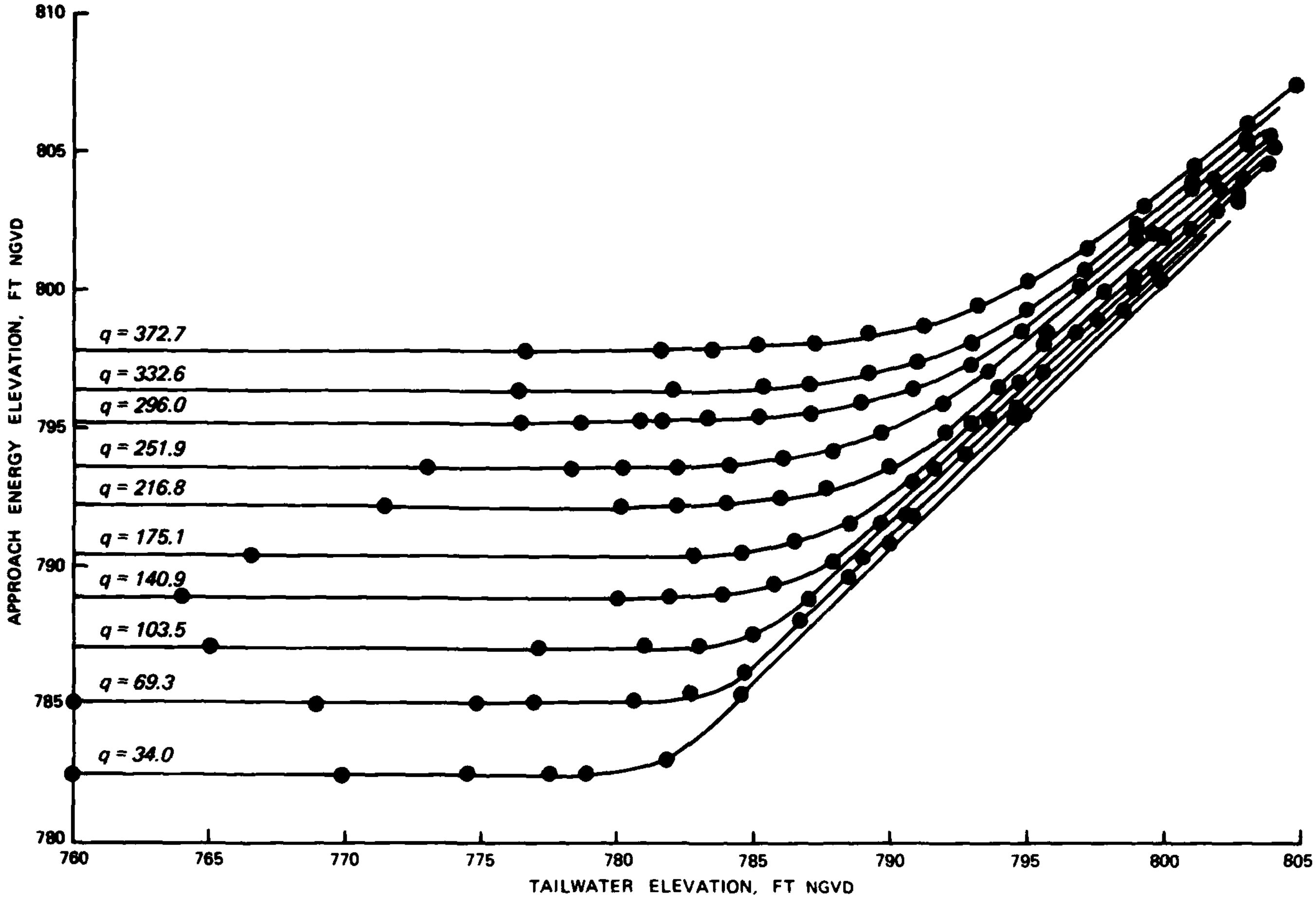


Q THOUSANDS OF CFS
28-FT APPROACH DEPTH
CREST LENGTH 460 0 FT

NOTE ABUTMENT COEFFICIENT FROM HYDRAULIC DESIGN CRITERIA

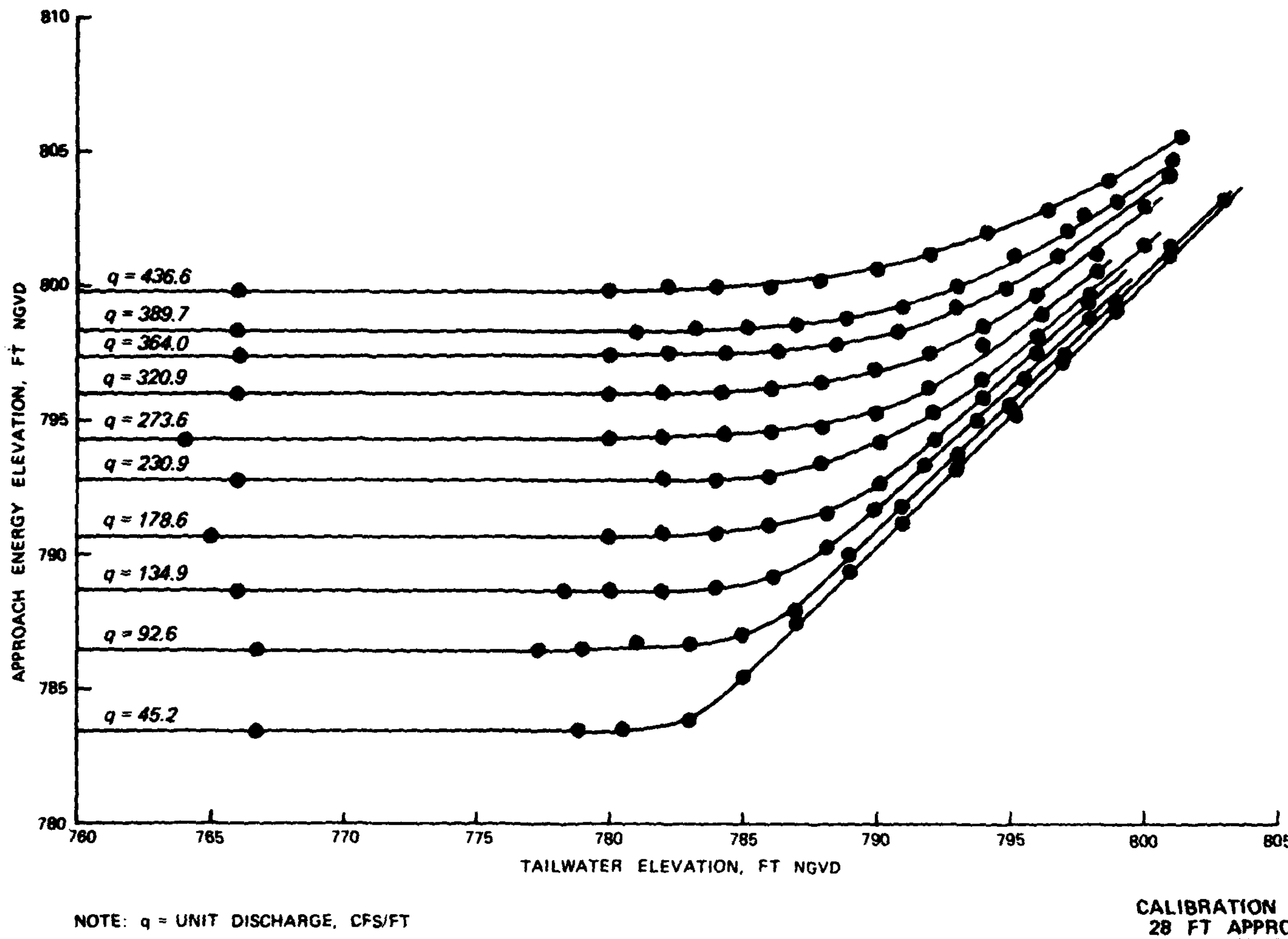
- ● COMPUTED WITH EXPANSIVE FLOW OVER LOCK WALL
- ■ MODEL TEST WITHOUT EXPANSIVE FLOW OVER LOCK WALL

RATING CURVE
DISCHARGE VS POOL ELEVATION FOR EXPECTED TAILWATER

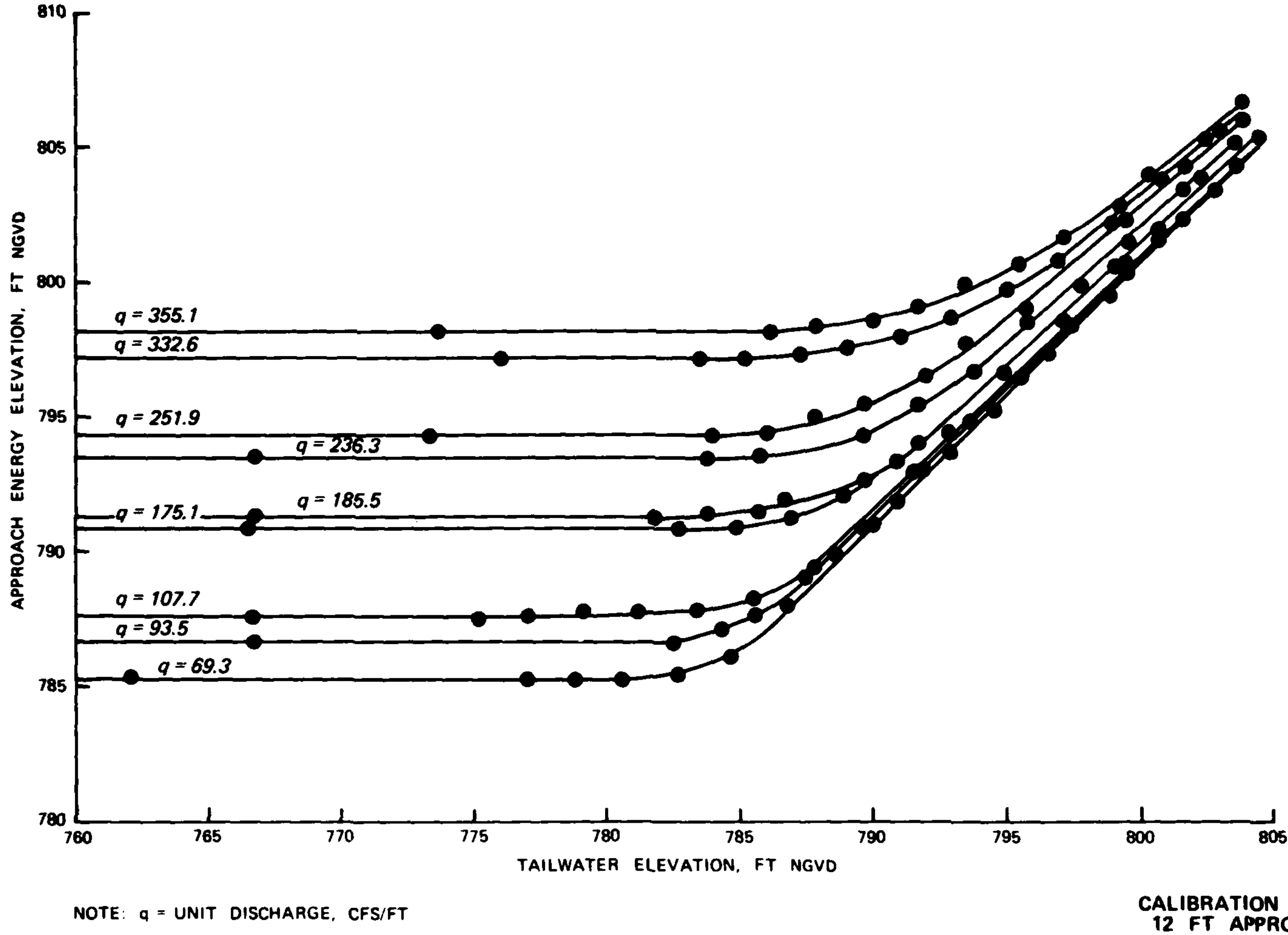


NOTE: q = UNIT DISCHARGE, CFS/FT

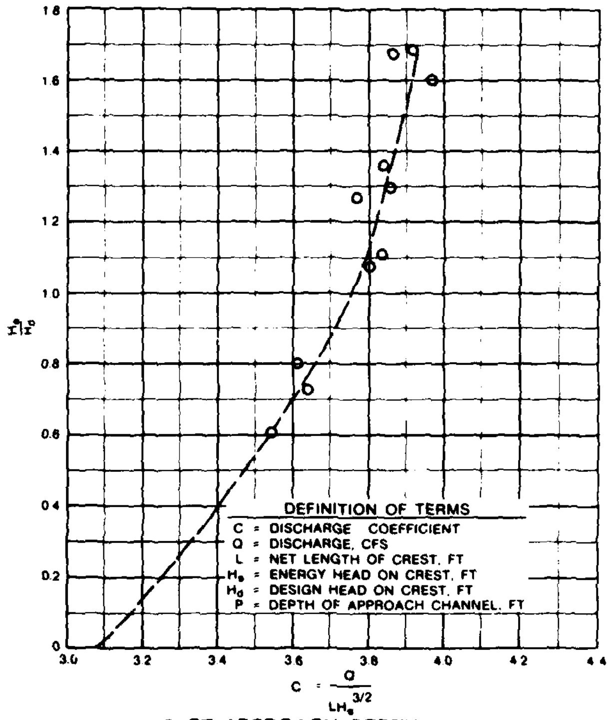
CALIBRATION DATA
28 FT APPROACH
q = 34.0 — 372.7 CFS/FT



CALIBRATION DATA
28 FT APPROACH
q = 45.2 — 436.6 CFS/FT

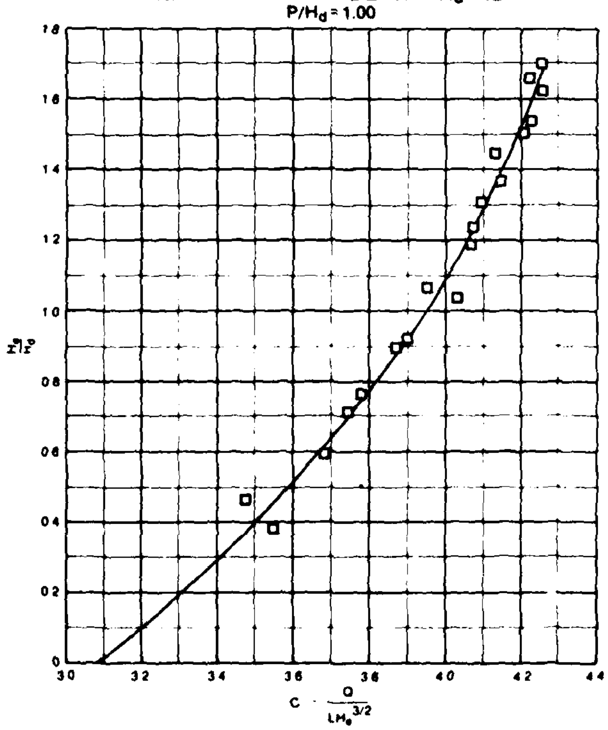


CALIBRATION DATA
12 FT APPROACH
q = 69.3 – 355.1 CFS/FT



12-FT APPROACH DEPTH $H_d = 12'$

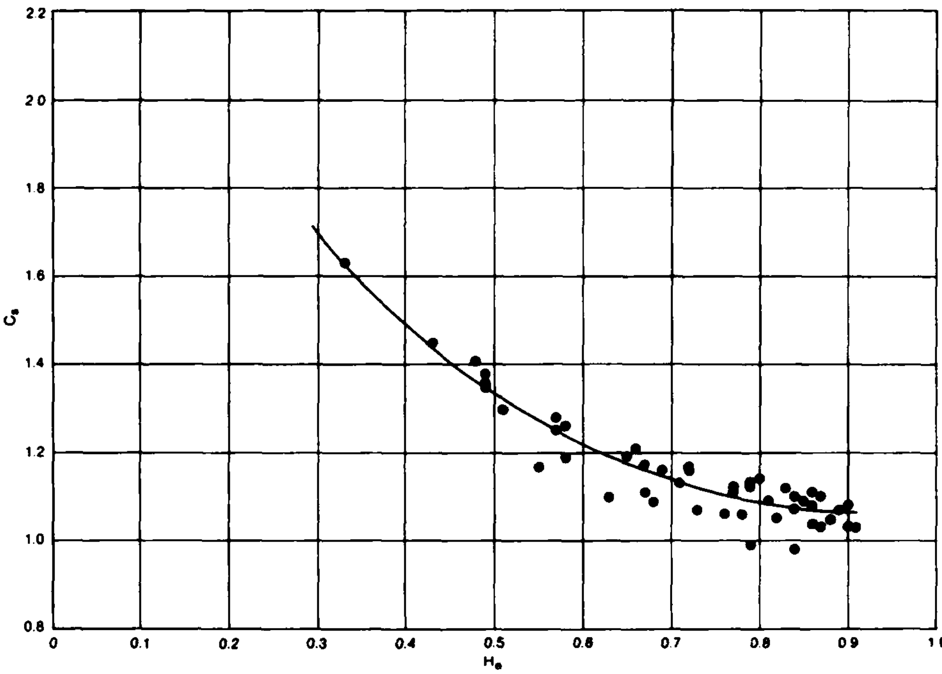
$P/H_d = 1.00$



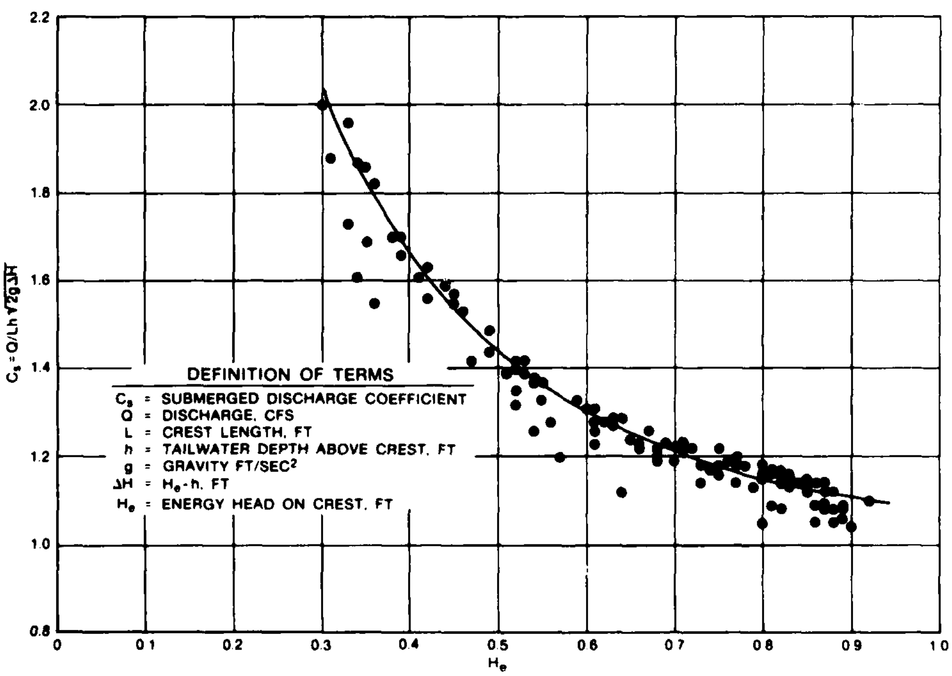
28-FT APPROACH DEPTH $H_d = 12'$

$P/H_d = 2.33$

FREE FLOW
DISCHARGE COEFFICIENTS

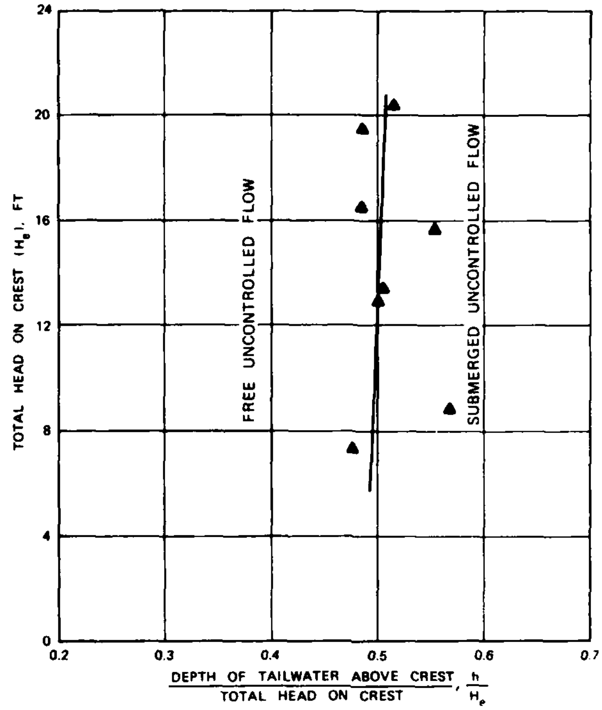


12-FT APPROACH DEPTH

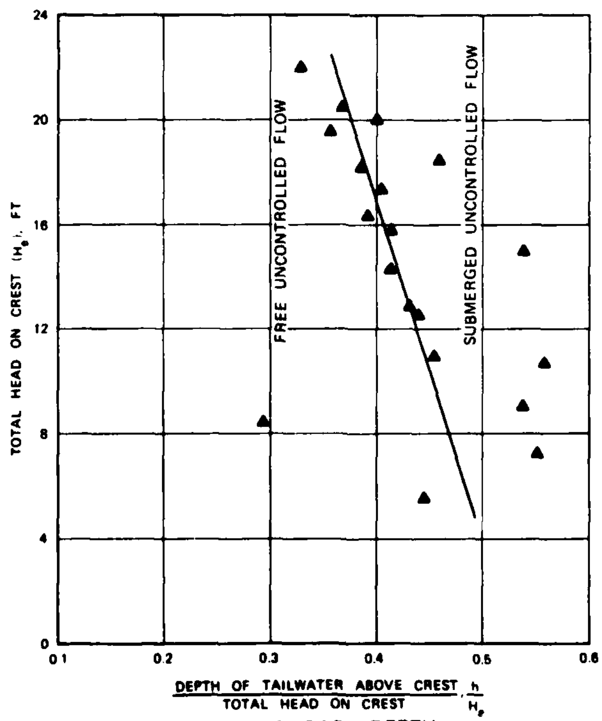


28-FT APPROACH DEPTH

SUBMERGED FLOW
DISCHARGE COEFFICIENTS



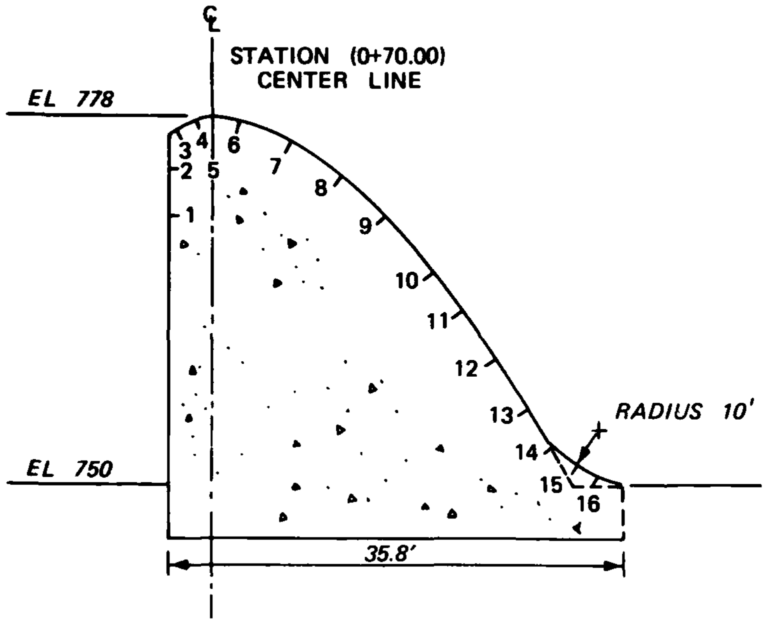
12-APPROACH DEPTH



28-APPROACH DEPTH

NOTE: ▲ POINT AT WHICH TAILWATER CHANGES ENERGY HEAD ON CREST BY 1%

UNCONTROLLED-FLOW REGIMES

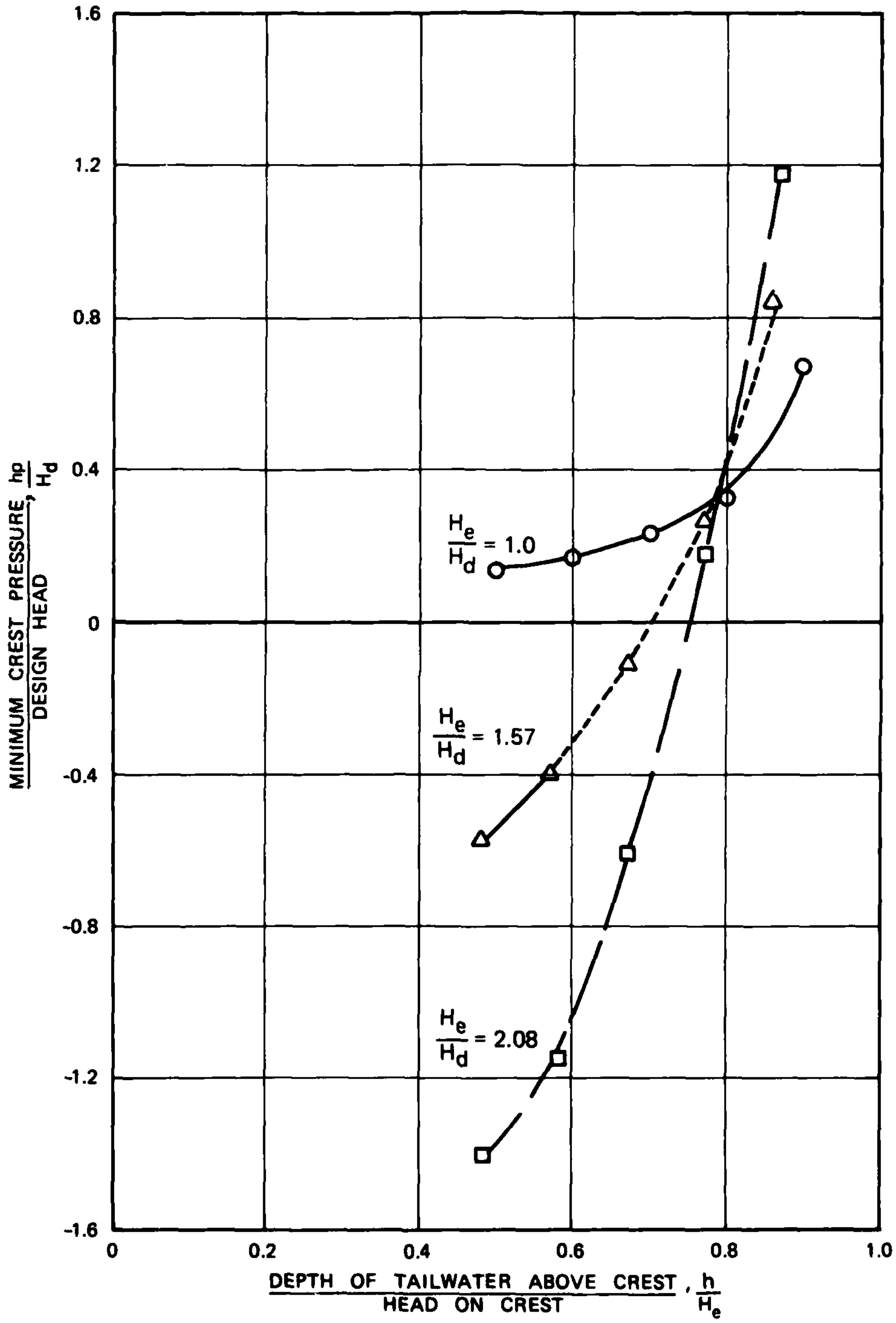


SPILLWAY CREST SHAPE

PIEZOMETER NUMBER	STATION	ELEVATION FT NGVD
1	0 + 73.36	768.0
2	0 + 73.36	773.0
3	0 + 72.90	776.0
4	0 + 71.00	777.8
5	0 + 70.00	778.0
6	0 + 68.00	777.6
7	0 + 65.00	777.0
8	0 + 60.20	774.0
9	0 + 57.00	771.0
10	0 + 53.80	767.5
11	0 + 50.50	763.0
12	0 + 47.70	759.0
13	0 + 45.00	755.0
14	0 + 43.40	753.0
15	0 + 42.00	752.0
16	0 + 39.50	750.5

NOTE: PIEZOMETER 5, IS LOCATED AT THE CENTER LINE (0+70.00) OF THE SPILLWAY CREST (EL 778.0)

PIEZOMETER LOCATIONS



RELATION BETWEEN CREST PRESSURES, DESIGN HEAD AND SUBMERGENCE FOR UPSTREAM QUADRANT

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Rothwell, Edward D.
Grays Landing Spillway and Stilling Basin, Monongahela River, Pennsylvania : Hydraulic model investigation / by Edward D. Rothwell, Noel R. Oswalt and Stephen T. Maynord (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; 1981.
22, [44] p., 12 p. of plates : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; HL-81-13)
Cover title.
"Prepared for U.S. Army Engineer District, Pittsburgh."
Final report.

1. Grays Landing Spillway (Pa.) 2. Hydraulic models. 3. Spillways. 4. Stilling basins. I. Oswalt, Noel R. II. Maynord, Stephen T. III. United States. Army. Corps of Engineers. Pittsburgh District. IV. U.S.

Rothwell, Edward D.
Grays Landing Spillway and Stilling Basin : ... 1981.
(Card 2)

Army Engineer Waterways Experiment Station. Hydraulics Laboratory. V. Title VI. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-81-13.
TA7.W34 no.HL-81-13

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